NIST GCR 21-917-48v3B

Seismic Design of Archetype Steel Buildings in Central and Eastern United States

Volume 3B – 3-story Education Building in St. Louis, Missouri Supplementary Documentation

Applied Technology Council

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**Seismic Design of Archetype Steel
Buildings in Central and Eastern
United States
Volume 3B – 3-story Education
Building in St. Louis, Missouri
Supplementary Documentation**

Prepared for
*U.S. Department of Commerce
Engineering Laboratory
National Institute of Standards and Technology
Gaithersburg, MD 20899-8600*

By
*Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, CA 94065*

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Participants

National Institute of Standards and Technology

John (Jay) Harris III, Research Structural Engineer
Earthquake Engineering Group, Materials and Structural Systems Division, Engineering
Laboratory
www.NEHRP.gov

Applied Technology Council

201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065
www.ATCouncil.org

Program Management

Jon A. Heintz (Program Manager)
Ayse Hortacsu (Associate Program
Manager)

Project Technical Committee

Donald Scott (Project Director)
John Hutton
Adrian Persaud

Project Review Panel

Ramon Gilsanz
Larry Griffis
Emily Guglielmo
Eric Hines
Erik A. Madsen

Working Group

Steve Antilla
McKell Bowen
Jared Dragovich
Hai Lin
Chris Putman
Cameron Prince
Gavin Rinaldo

Preface

Seismic design codes in the United States have evolved since first introduced in 1925; however, they are primarily based on the knowledge of performance requirements for buildings located in high seismicity regions of the United States, such as the West Coast. These model codes are extrapolated for use in areas with moderate seismicity, such as the Central and Eastern United States (CEUS), where member sizes of the lateral-force-resisting systems may be governed by wind requirements. There is a need to understand the seismic performance of buildings when the controlling design load is from wind effects.

In September 2018, the Applied Technology Council (ATC) commenced a task order project under National Institute of Standards and Technology (NIST) Contract SB1341-13-CQ-0009 to develop designs for archetype steel buildings to facilitate future research in understanding the seismic performance of buildings when the controlling design load is from wind. For this purpose, three archetype steel buildings were designed in accordance with older building codes and current building codes for specific locations within the CEUS. This document is one of three volumes presenting representative structural calculations supplementing design drawings for archetype steel buildings provided NIST GCR 21-917-48v3A.

The designs presented were developed by design firms PCS Structural Solutions of Seattle, Washington and Uzun + Case of Atlanta, Georgia. The Project Technical Committee, consisting of Don Scott, John Hutton, and Adrian Persaud monitored and guided the technical efforts of the Project Working Groups, which included Steve Antilla, Jared Dragovich, Hai Lin, Chris Putman, Cameron Prince, and Gavin Rinaldo. Project Working Group member McKell Bowen led the development of two of the three designs presented. The Project Review Panel, consisting of Melissa Burton (ATC Board Contact), C.B. Crouse, Ramon Gilsanz, Larry Griffis, Emily Guglielmo, Eric Hines, and Erik Madsen provided technical advice and consultation over the duration of the work. The names and affiliations of all who contributed to this report are provided in the list of Project Participants in the main Volume.

ATC also gratefully acknowledges Jay Harris (Contracting Officer's Representative) for his input and guidance throughout the project development process. ATC staff member Justin Moresco and Ginevra Rojahn provided project management support and report production services, respectively.

Ayse Hortacsu
Associate Program Manager

Jon Heintz
Program Manager

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Design Case A

1987 NBC “Risk Category II” “Seismic Design Category C”

This section presents representative structural calculations for Building 3 (3-story education facility in St. Louis, Missouri) Design Case A. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v3A.

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DESIGN CRITERIA

DESIGN CRITERIA

VERTICAL LOADS

AREA	DESIGN DEAD LOAD	LIVE LOAD	CONCENTRATED LOADS
CLASSROOM AND OFFICE	10 PSF	50 PSF	1000#
CORRIDORS ABOVE 1ST FLOOR	10 PSF	80 PSF	1000#
ROOF	20 PSF	20 PSF	-

SNOW:

$P_g = 20 \text{ PSF} = \text{GROUND SNOW LOAD}$

$P_f = C_e I P_g = \text{FLAT ROOF SNOW LOAD} = 14 \text{ PSF}$, USE 20 FOR CALCULATION

$I = 1.0$, $C_e = 1.0$

LATERAL FORCES

LATERAL FORCES ARE TRANSMITTED BY DIAPHRAGM ACTION OF ROOF AND FLOORS TO BRACED FRAME/MOMENT FRAME. LOADS ARE THEN TRANSFERRED TO FOUNDATION BY BRACED FRAME/MOMENT FRAME ACTION WHERE ULTIMATE DISPLACEMENT IS RESISTED BY PASSIVE PRESSURE OF EARTH AND/OR SLIDING FRICTION. OVERTURNING IS RESISTED BY DEAD LOAD OF THE STRUCTURE.

WIND:

THE BUILDING MEETS THE CRITERIA PER NBC 1987 SECTION 1112.0.

- EXPOSURE CATEGORY = B
- BASIC WIND SPEED, $V = 70 \text{ MPH}$
- EQUIVALENT RISK CATEGORY PER TABLE 1.5-1 = II
- PRESSURE COEFFICIENT (ENCLOSED) = 0.8, -0.5
- WIND IMPORTANCE FACTOR $I_w = 1.0$
- BRACE FRAME BASE SHEAR $V = 78.6 \text{ KIPS}$; MOMENT FRAME BASE SHEAR $V = 55.5 \text{ KIPS}$

SEISMIC: (NBC 1987) $V = Z I_k C_{SW}$

$Z = 3/8$

SEISMIC IMPORTANCE FACTOR, $I = 1.0$

EQUIVALENT RISK CATEGORY PER TABLE 1113.1 = II

$k = 1.0$ AT BRACED FRAME, $k = 1.0$ AT MOMENT FRAME

W = EFFECTIVE SEISMIC WEIGHT OF BUILDING = 2340 KIPS

ANALYSIS PROCEDURE USED = EQUIVALENT LATERAL FORCE PROCEDURE

DESIGN BRACE FRAME BASE SHEAR $V = 123 \text{ KIPS}$

DESIGN MOMENT FRAME BASE SHEAR $V = 115.3 \text{ KIPS}$

FOUNDATION DESIGN CRITERIA

SOIL BEARING PRESSURE: 4000 PSF (ASSUMED)

ACTIVE PRESSURE - RESTRAINED: 50 PCF +14H SEISMIC SURCHARGE (ASSUMED)

ACTIVE PRESSURE - UNRESTRAINED: 35 PCF +6H SEISMIC SURCHARGE (ASSUMED)

PASSIVE RESISTANCE: 200 PCF (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

COEFFICIENT OF FRICTION: .35 (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

*1/3 INCREASE ALLOWED FOR SEISMIC OR WIND LOADING

CONCRETE

CAST-IN-PLACE CONCRETE

ITEM	DESIGN f'_c (PSI) (AT 28 DAYS U.N.O.)
FOUNDATIONS	3000
SLABS ON GRADE AND SLABS ON METAL DECK	4000

REINFORCING STEEL

ASTM A615, GRADE 60 TYPICAL UNLESS NOTED OTHERWISE.

REINFORCING SPLICE AND DEVELOPMENT LENGTH SCHEDULE, $F_y=40$ KSI (UNLESS NOTED OTHERWISE)

BAR SIZE	MINIMUM LAP SPLICE LENGTHS ("Ls")	MINIMUM DEVELOPMENT LENGTHS ("Ld")
#3	1'-6"	1'-3"
#4	2'-0"	1'-7"
#5	2'-7"	2'-0"
#6	3'-1"	2'-4"
#7	4'-6"	3'-6"
#8	5'-2"	3'-11"
#9	5'-10"	4'-6"
#10	6'-6"	5'-0"
#11	7'-3"	5'-7"

STRUCTURAL STEEL

MATERIAL PROPERTIES

WIDE FLANGE SECTIONS: ASTM A36 ($F_y = 36$ KSI)

OTHER SHAPES AND PLATES: ASTM A36 ($F_y = 36$ KSI)

STRUCTURAL TUBING: RECTANGULAR & SQUARE - A500 GRADE BB ($F_y = 46$ KSI)

MACHINE BOLTS (M.B.): ASTM A307

HIGH-STRENGTH BOLTS: A325-ASTM F1852, A490-ASTM F2280

ANCHOR BOLTS (A.B.): ASTM A490, GRADE 36, UNLESS OTHERWISE NOTED

Design Criteria

Job Site: St. Louis, MO

Design Code: BOCA 1987, ANSI A58.1-1982

Ground Snow Load: 20 psf [ANSI A58.1] (See attached)

Minimum Allowable bearing pressure: 3000 PSF [BOCA Table 1201 Presumptive surface bearing value for soft broken shale, soft clay, assumed] (See attached)

Seismic: Zone 2 (See attached)

Building Classification: Category I in ANSI A58.1-1987

$$I := 1.0$$

Roof Snow Load:

$$C_e := 1.0 \quad C_t := 1.0$$

$$p_g := 20 \quad \text{psf}$$

$$P_f := 0.7 \cdot C_e \cdot C_t \cdot I \cdot p_g = 14 \quad \text{psf}$$

Use 20 psf for snow load

Floor Live load:

Classroom=40 psf

Office=50 psf (use Office Load)

Dead Load:

Roof:

Roofing=3 psf

Roof Deck=2.5 psf

Steel Frame=8 psf

MEP=3 psf

MISC=3 psf

Floor:

Floor Finish=3 psf

Steel Frame=10 psf

Floor Deck(W3)+Concrete Topping(2.5" Normal)=48 psf

MEP=3 psf

MISC=3 psf

Partition=20 psf

$$\text{TotalLoad} := 3 + 2.5 + 8 + 3 + 3 = 19.5 \quad \text{psf}$$

Use 20 psf to design

$$\text{FloorLoad} := 3 + 10 + 48 + 3 + 3 + 20 = 87 \quad \text{psf}$$

Use 90 psf to design

GRAVITY DESIGN

ROOF BEAM DESIGN

Beam # = RB4

Shape W 16x26

Properties	Geometry	
	A	7.67 in. ²
	d	15.65 in.
	b _f	5.5 in.
	t _f	0.345 in.
	t _w	0.25 in.
	W(weight)	26 #/ft.
	r _T	1.38
	d/A _f	8.25

Compact Section Criteria	
b _f /2t _f	7.97
F _y	42.9
d/t _w	62.6
F _y	43.3 ksi
F _y	16.9 ksi

Elastic Properties					
X-X			Y-Y		
I _x	300	in. ⁴	I _y	9.59	in. ⁴
S _x	38.3	in. ³	S _y	3.49	in. ³
r _x	6.25	in.	r _y	1.12	in.
Plastic Modulus					
Z _x	44	in. ³	Z _y	5.48	in. ³

Torsional Constant	
J	0.261 in. ⁴

Warping Constant	
C _w	562 in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Note:

Input/ verify data in yellow box:

Non-Composite Beam Design AISC Seventh Edition

Building info:

Steel Properties		Loading	
Beam Length =	29.33 ft.	Dead Load (D)=	20 psf
Beam Spacing (w) =	10 ft.	Live Load(L)=	0 psf
Unbraced Length =	29.33 ft.	Roof Live Load(Lr)=	20 psf
Fy =	36 ksi	Snow Load(S)=	20 psf

Deflection Criteria		
Total Load	L/360=	0.98 in.

Max Moment:	
MD(Max Dead)=	258.07 kips-in
MD(Dead Overwrite)=	0.00 kips-in
MD(Max Use)=	258.07 kips-in
ML(Max Live)=	0.00 kips-in
ML(Live Overwrite)=	0.00 kips-in
ML(Max Use)=	0.00 kips-in
M(Lr or S)(Max Live)=	258.07 kips-in
M(Lr or S)(Live Overwrite)=	0.00 kips-in
M(Lr or S)(Max Use)=	258.07 kips-in
Mu=	516.15 kips-in

Controlling Case = D+L+(Lr or S)

Max Shear	
VD(Max Dead)=	2.93 kips
VD(Dead Overwrite)=	0.00 kips
VMD(Max Use)=	2.93 kips
VL(Max Live)=	0.00 kips
VL(Live Overwrite)=	0.00 kips
VL(Max Use)=	0.00 kips
V(Lr or S)(Max Live)=	2.93 kips
V(Lr or S)(Live Overwrite)=	0.00 kips
V(Lr or S)(Max Use)=	2.93 kips
Vu=	5.87 kips

Controlling Case = D+L+(Lr or S)

Solution Summary:		
Try	W 16x26	<---- Change W shapes
Member	Checked Item	DCR
	Beam Bending	56%
	Beam Shear	10%
	Beam deflection	79%
Overall Check		79%

Typical Bending

Camber =	0 in.
----------	-------

Non-Composite Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced lenth (Lb)= 29.33 ft

Checks for Typical Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f/2t_f \leq 55.2/\sqrt{F_y}$	7.97	8.7	Good
$d/t_w \leq 412/\sqrt{F_y}$	62.6	68.7	Good
$l_b/\leq 76b_f/\sqrt{F_y}$	351.96	69.7	No Good, Use Alternate EQN
$l_b/\leq \frac{20,000}{(d/A_f) F_y}$	351.96	67.4	No Good, Use Alternate EQN
Use Alternate Equation			

Checks for Alternate Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2/\sqrt{F_y} < b_f/2t_f$	7.97	8.7	Good
$b_f/2t_f \leq 95.0/\sqrt{F_y}$	7.97	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_b = 0.66F_y$

Fb= N/A ksi

Alternate Bending Stress Equation: $F_b = F_y [0.733 - 0.0014(\frac{b_f}{t_f})\sqrt{F_y}]$

Fb= 24 ksi

Shear Stress Equation: $F_v = 0.40F_y$

Fv= 14.4 ksi

Calculate Stresses in Beam

Bending Stress:	fb= Mu/Sx=	13.5	ksi	Beam Okay
Shear Stress:	fv= Vu/(tw*d)=	1.5	ksi	Beam Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0.77	in.	Beam Okay

FLOOR BEAM DESIGN

Beam # = FB4

Shape W 16x36

Properties	Geometry	
	A	10.6 in. ²
	d	15.86 in.
	b _f	6.985 in.
	t _f	0.43 in.
	t _w	0.295 in.
	W(weight)	36 #/ft.
	r _T	1.79 in.
	d/A _T	5.28 in.

Compact Section Criteria	
b _f /2t _f	8.1
F _y	64
d/t _w	53.8
F _y	- ksi
F _y	22.9 ksi

Elastic Properties			
X-X		Y-Y	
I _x	448 in. ⁴	I _y	24.5 in. ⁴
S _x	56.5 in. ³	S _y	7 in. ³
r _x	6.51 in.	r _y	1.52 in.
Plastic Modulus			
Z _x	64 in. ³	Z _y	10.8 in. ³

Torsional Constant	
J	0.54 in. ⁴

Warping Constant	
C _w	- in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Note: Input/ verify data in yellow box:

Composite Beam AISC 8th Edition

Note: The following calculation is basic on the "Steel Tips by Steel Committee of California" dated Jan 1987 and of Composite Design for building construction in AISC 8ed Example 2

Assumption:

Item	Ribs Perpendicular	Ribs Parallel
1. Concrete Area below Top of deck	Neglect	Include
2. Stud Reduction Factor	$\frac{0.85}{(Nr)^{1/2}} \left(\frac{wr}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right)$	$0.6 \left(\frac{wr}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right)$
3. Maximum Stud Spacing	32 in.	Not Specified
4. Deck Welding	16 in.	Not Specified
5 Minimum Width of Rib	2 in.	Depends on Nr

Building info:

Steel			Concrete			Deck		
Beam Length	29.3	ft.	Weight	145	pcf	Depth hr	3	in.
Beam Spacing	10	ft.	Thickness t	5.5	inches	Conc. thickness tc	2.5	in.
Fy	36	ksi	f'c	3000	psi	Stud Dia.	3/4	in.
			E'c	3122	ksi	Stud Height	4.5	in.
			β ₁	0.85		Deck Orientation	Perpendicular	
			Light Weight concrete Reduction	1		Deck Width, wr	4.5	in.

Uniform Load		Deflection Criteria	
Dead Load	90 psf	Dead Load	L/240 or 1.5 in.
Live Load	50 psf	Live Load	L/360
Roof Live Load	0 psf		
Snow Load	0 psf		

Uniform Load W/ Combo	
w(D+L+Lr or S)	140 psf

Max Moment:	kips-in	Kips-ft
M(max D)	1159.0	96.6
M(D overwrite)	0.0	0.0
M(Dead use)	1159.0	96.6
M(max D+L)	1802.8	150.2
M(D+L overwrite)	0.0	0.0
M(max use)	1802.8	150.2
Max Shear	kips	
V(max D)	13.185	
V(max D+L)	20.51	
V(D+L overwrite)	0	
V(max use)	20.51	

Solution Summary:

W 16x36 <---- Change W shapes

Partial Composite: **75%**

ASD Result:

DCR	
Concrete Stress	0.168
Steel Stress	0.855
Dead Load Δ	0.790 1.1567 in.
Live Load Δ	0.209 0.2044 in.
Shoring?	Need Shoreing <----Check this
# of studs	20 at each side of max moment
Studs per rib check	OK
Camber=	0.750 in.
Overall	GOOD

Result illustration:

Beam	Stud	Camber
W 16x36	40	0.750
o Δ		
20.51 Reaction		20.51 Reaction

$n=E/E_c$	9
-----------	---

1. Effective Width :

$L/4$	87.9	in.
s	120	in.
$16t+bf$	95.0	in.

Govern b_{eff} : 87.9 in.**2. Required Section Moduli**

$M(D+L+S), S_{tr}$	75.1	in.^3
$M(D \text{ only}), S_s$	48.3	in.^3

3. Composite design section properties**a. Moment of Inertia**

$Ac_{\perp}$	219.75	
$Ac_{//}$	357.555	
$Ac=b(tc)$	219.75	in.^2
$A'c=Ac/n$	24.42	in.^2
$Y_s=d/2$	7.93	in.
$Y_c=d+hr+tc/2$	20.11	in.

Section:	A	Y	AY	ds	Ad^2	I_o
W 16x36	10.6	7.93	84.1	8.49	764.6	448.0
Concrete	24.41667	20.11	491	3.69	331.9	12.7
Sum	35.01667		575.1		1096.5	460.7
					$I_{tr}=$	1557.23 in.^4

 $Y_b=$ 16.42 in.**b. Section Moduli**

			Partial Composite	
Str=	94.82	in.^3	Seff=	89.7 in.^3
St=	315.42	in.^3	leff=	1408.6 in.^4

4. Check Stress

a. Concrete				
$f_c=M/n*St$	0.2268	ksi <	1.35	OK
b. Steel				
Total Load, Str=	89.7	ksi >	75.117875	OK
Dead Load, Ss=	56.5	ksi >	48.29	OK
Web shear, Fv=	4.384	ksi <	20	OK

5. Check Deflection

$\Delta = \frac{5wL^4}{384 * EI} = \frac{ML^2}{1920 I}$				
$\Delta D=$	1.156711	in. <	1.5	OK
$\Delta L=$	0.20438	in. <	1.0	OK

6. Check if shore is required

Str max=	87.26	in.^3 >	94.82	Need Shoreing
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7. Shear Connection, Stud

a. Max Stud dia	1.075	in. >	0.75	OK
-----------------	-------	-------	------	----

b. Horizontal shear

$V_h=0.85*f_c*Ac/2$	280.1813	kips	Concrete	
$V_h=As*F_y/2$	190.8	kips	Steel	<----- Govern

Reduction Factor

$$\frac{0.85}{(Nr)^{1/2}} \left(\frac{w_r}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right) \quad \text{or} \quad 0.6 \left(\frac{w_r}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right)$$

$$Rf_{\perp}= 0.64$$

$$Rf_{//}= 1.000$$

$$Rf(USE)= 0.638$$

$$q \text{ per stud} = 11.5000 \quad \text{<-Lightweight Concrete reduction included}$$

$$q*Rf= 7.331$$

Stud Shear	F'c(From AISC V8 Table 1.11.4) (ksi)		
Connector	3000	3500	4000
0.5	5.1	5.5	5.9
0.625	8	8.6	9.2
0.75	11.5	12.5	13.3
0.875	15.6	16.8	18

Stud Per Rib, Nr	1	
Rib Spacing	6	in.

Number of Studs= 20 At each side of where max moment is

Check Stud Per Rib	20	ft	<	29.3	OK
--------------------	----	----	---	------	----

COLUMN DESIGN

COLUMN B5

Column ID	Roof Load (Kips)		Third Floor		Second Floor		Force at the base
1	0	0	0	0	Fba''+Fb8	12.0	12.0
2	Rba+RB1	11.46	Fba+FB1	50.12	FB8+Fba'	42.4	104.0
3	RBd+RB1	9.17	FBd+FB1	39.64	FB1	20.1	68.9
4	RBb+RB6	18.51	FB4+FBg	53.42	53.42	53.4	125.3
5	Rbi+RB6	14.5	FB4+Fba(Canti)	52.11	52.11	52.1	118.7
6	0		0	0.00	Fba''+Fba''	14.7	14.7
7	Rba+Rba+RB1	17.19	Fba+Fba+FB1	80.15	FB8+Fba'+Fba'+FB1	100.3	197.7
8	RBd+Rbe+RB1+RB2	18.05	Fbd+Fbe_Fb1+FB2	87.36	87.36	87.4	192.8
9	Rbe+RB4	13.44	Fbe+FB4+FBg	93.32	93.32	93.3	200.1
10	RB4+Rbi+Rba	20.7	Fba'+Fba'+FB4	80.57	80.57	80.6	181.8
11	0	0	0	0.00	Fba''+Fbb'+FB8	39.3	39.3
12	Rba+RBb+RB1	14.9	Fba+FBb+FB1	72.45	FB8+Fba'+FBb'+FB1	89.9	177.2
13	RB1+Rbe+RBg+RB2	24.55	FB1+Fbe+FBg+FB2	100.72	100.72	100.7	226.0
14	RB1+Rbe+RBg+RB2	22.55	FB2+Fbe+FBg+FB4	101.14	101.14	101.1	224.8
15	Rba+RBb+RB4	15.04	FB4+Fba+FBb	72.87	72.87	72.9	160.8
16	RB1+RBg+RBg+RB2	26.39	FB1+FBg+FBg+FB2	93.73	93.73	93.7	213.9
17	0	0	0	0.00	FBb''+FBb''+FB8	15.4	15.4
18	RBb+RBb+RB1	12.61	FBb+FBb+FB1	64.75	FB8+Fba'+FBb'+FB1	89.9	167.2
19	RB2+RBg+RBg+RB4	26.53	FBg+FBg+FB2+FB4	94.15	94.15	94.2	214.8
20	RB4+RBb+RBb	12.75	FB4+FBb+FBb	65.17	65.17	65.2	143.1
21	RB1+RBg+Rbe+RB2	24.7	FB1+FBg+Fbe	92.9	92.9	92.9	210.5
22	0	0	0	0	FBb''+FBb''+FB8	15.4	15.4
23	RBb+RBc+RB1	12.61	FBb+FBb+FB1	64.75	FB8+FBb'+FBb'	59.3	136.7
24	RB2+RBg+RBg+RB4	26.53	FB2+FBg+FBg+FB4	94.15	94.15	94.2	214.8
25	RBb+RBc+RB4	12.75	FB4+FBb+FBc	65.17	65.17	65.2	143.1
26	RBc+RB1	9.17	Fbi+FB5	49.32	FBb''+FB8	10.0	68.5
27	RBc+RB(7-ED)	10.22	FBb+FBk	41.63	FB8+FBb'+FBb'+FB1	79.4	131.3
28	RB(7-ED)+Rbe+Rbf+RB2	21.02	FBK+Fbe+FBf+FB2	100.35	86.474	87.5	208.8
29	RBg+RBh+RB4	21.15	FB2+FBg+FBf+FBk	90.84	90.84	90.8	202.8
30	RB4+RBc+RBg	15.18	FBk+FBc+FBf	72.44	72.44	72.4	160.1
31	0	0	0	0	FBb'+FB1	47.5	47.5
32	0	0	0	0	FBb'+FBb'	54.7	54.7
33	RBg+RBg	11.74	FB3+FBf	41.15	31.464	31.5	84.4
34	0	0	FB6	4.29	0	0.0	4.3
35	RB(8-EC)+Rbf	8.91	FB6+FBj+FB3	49.14	0	0.0	58.1
36	RB(8-EC)+RBh+RB(8-C-B.5)	11.51	FBx+FBf	40.81	33.56	33.6	85.9
37	RB(8-C-B.5)+RB(8-B.5-B)	2.1	FB3+ FBf	41.15	31.464	31.5	74.7
38	RB(8-B.5-B)+RBg	7.33	2Kips	2.5	2.5	2.5	12.3

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v3B>

Member # = B5

Shape W 10x54

Properties	Geometry	
	A	15.8 in. ²
	d	10.09 in.
	b _f	10.03 in.
	t _f	0.615 in.
	t _w	0.37 in.
	W(weight)	54 #/ft.
	r _T	2.75
	d/A _f	1.64

Compact Section Criteria	
b _f /2t _f	8.2
F _y	63.5 ksi
d/t _w	27.3
F _y	— ksi
F _y	— ksi

Elastic Properties				
X-X			Y-Y	
I _x	303	in. ⁴	I _y	103 in. ⁴
S _x	60	in. ³	S _y	20.6 in. ³
r _x	4.37	in	r _y	2.56 in
Plastic Modulus				
Z _x	66.6	in. ³	Z _y	31.3 in. ³

Torsional Constant	
J	1.82 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	13.75 ft.
Beam Spacing (w) =	0 ft.
Tributary Area to Col=	0 ft.^2
K=	1
Unbraced Length =	13.75 ft.

Loading	
Dead Load (D)=	90 psf
Live Load(L)=	50 psf
Roof Live Load(Lr)=	0 psf
Snow Load(S)=	25 psf

Eccentricity on Column	
ex =	2 in.
ey =	0 in.

Deflection Criteria		
Total Load	L/360=	0.46 in.

Max Moment About Major Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Shear	
VD(Max Dead)=	0 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	0 kips
VL(Max Live)=	0 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	0 kips
V(Lr or S)(Max Live)=	0 kips
V(Lr or S)(Live Overwrite)=	0 kips
V(Lr or S)(Max Use)=	0 kips
Vu=	0 kips

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	0 kips
Pu=	200 kips

Solution Summary:	
Try	W 10x54 <---- Change W shapes

Member	Major Axis Beam Bending	0%
	Beam Shear	0%
	Minor Axis Beam Bending	0%
	Axial	75%
	Deflection	0%
	Combined Bending and Axial	75%
Overall Check		75%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 13.75 ft
K= 1
KL= 13.75 ft
KL/r= 64.5

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 17 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 12.7$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (Lb)= 13.75 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	8.2	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	27.3	12.2	No Good, Use Alternate EQN
$l_b \leq 76 b_f \sqrt{F_y}$	165	127	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d / A_f) F_y}$	165	339.6	Good
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	8.2	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	8.2	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$$F_{bx} = \text{N/A} \text{ ksi}$$

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$$F_{bx} = 23.9 \text{ ksi}$$

Shear Stress Equation: $F_v = 0.40 F_y$

$$F_v = 14.4 \text{ ksi}$$

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$$F_{by} = 27 \text{ ksi}$$

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	0	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	0	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.75	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.59	Less than 1.0, Okay

Column Name: **B5**

All Equations can be found on Page 3-99 of AISC 8th edition

Column= **W 10x54**

Column Properties	
<i>b</i>	10.03 in
<i>d</i>	10.09 in

Material Properties	
Footing <i>f'_c</i> =	3 ksi
<i>F_y</i> (Plate)=	36 ksi

Max Axial Load = **200** kips

Required Base Plate Area =	$A_{pl} = \frac{P_a}{0.375 f'_c} =$	177.8	in ²
----------------------------	-------------------------------------	-------	-----------------

Base Plate Design	
<i>B</i> =	12 in.
<i>N</i> =	12 in.

Actual Bearing Pressure =	$f_p = \frac{P_a}{B * N} =$	1.4	ksi
---------------------------	-----------------------------	-----	-----

Plate Thickness		
$m = \frac{N - 0.95d}{2} =$	1.20725	in
$n = \frac{B - 0.8b}{2} =$	1.988	in
$t = m \sqrt{\frac{f_p}{0.25 F_y}} \leq n \sqrt{\frac{f_p}{0.25 F_y}} =$	0.78	in.

- P* = Total column load, kips
- A* = *B* × *N* = Area of plate, square inches
- F_b* = Allowable bending stress in base plate, ksi
- F_p* = Allowable bearing pressure on support, ksi
- f_p* = Actual bearing pressure
- f'_c* = Compressive strength of concrete, ksi or psi
- t* = Thickness of plate, inches

LATERAL DESIGN

EARTHQUAKE LOAD

St. Louis. located at the **Zone 2** of the Figure 1113.1 in BOCA 1987

Per BOCA 1987 1113.4

$$V := Z \cdot I \cdot K \cdot C \cdot S \cdot W$$

Dead Load

Wal Load $DL_w := 20 \text{ psf}$

Consist of
CFS wall (8psf) +
1.25" CMU Cladding(12psf)

Diaphragm 1:

Floor Dead Load $DL_1 := 90 \text{ psf}$

Floor Area $FloorA_1 := 9860 \text{ sf}$

Building Perimeter $P_1 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$

Wal Height $H_1 := \frac{(Height2 + Height1)}{2} = 13.75 \text{ ft}$

$W_1 := (DL_1 \cdot FloorA_1 + P_1 \cdot H_1 \cdot DL_w) = 1.025 \times 10^6 \text{ lb}$

Diaphragm 2:

Floor Dead Load $DL_2 := 90 \text{ psf}$

Floor Area $FloorA_2 := 10100 \text{ sf}$

Building Perimeter $P_2 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$

Wal Height $H_2 := \frac{(Height2 + Height3)}{2} = 14.125 \text{ ft}$

$W_2 := DL_2 \cdot FloorA_2 + P_2 \cdot H_2 \cdot DL_w = 1.051 \times 10^6 \text{ lb}$

Diaphragm 3(ROOF):

Floor Dead Load $DL_3 := 20 \text{ psf}$

Floor Area $FloorA_3 := 9560 \text{ sf}$

Building Perimeter $P_3 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$

Wal Height $H_3 := \frac{(Height3)}{2} = 7.25 \text{ ft}$

$W_3 := DL_3 \cdot FloorA_3 + P_3 \cdot H_3 \cdot DL_w = 2.639 \times 10^5 \text{ lb}$

$W := W_1 + W_2 + W_3 = 2.34 \times 10^6 \text{ lb}$

N-S:

Brace Frames are used as Lateral Force-Resisting System.

$$\begin{aligned}
 I &:= 1 && \text{Table 1113.1} \\
 Z &:= \frac{3}{8} && \text{For Location at Zone 2, Per Table 1113.4.1} \\
 K &:= 1.0 && \text{For Brace Frame as LFRS, Per Table 1113.4.3} \\
 h_n &:= 42.25 \text{ ft} && \text{Building Height} \\
 D_s &:= 39.3 \text{ ft} && \text{Shearwall/Brace Frame dimension parallel to applied force} \\
 T &:= 0.05 \cdot \frac{h_n}{\sqrt{D_s}} = 0.337 && \text{Fundamental period, Per 1113.4.5.2, Eq 2} \\
 C &:= \min\left(\frac{1}{15 \cdot \sqrt{T}}, 0.12\right) = 0.115 && \text{Numerical coefficient Per 1113.4.4 C factor} \\
 S &:= 1.5 && \text{Per 1113.4.6, assume soil profile=S3} \\
 CS &:= C \cdot S = 0.172 \\
 CS &:= \min(C \cdot S, 0.14) = 0.14 && \text{Per 1113.4.4 C factor CS limit} \\
 V_{\text{seismicNS}} &:= Z \cdot K \cdot I \cdot CS \cdot W = 1.228 \times 10^5 && \text{lb}
 \end{aligned}$$

E-W:

Ordinary Moment Frames are used as Lateral Force-Resisting System.

$$\begin{aligned}
 I &:= 1 && \text{Table 1113.1} \\
 Z &:= \frac{3}{8} && \text{For Location at Zone 2, Per Table 1113.4.1} \\
 K &:= 1.0 && \text{For Moment Frame as LFRS, Per Table 1113.4.3} \\
 h_n &:= 42.25 \text{ ft} && \text{Building Height} \\
 D &:= 147.5 \text{ ft} && \text{Building dimension parallel to applied force} \\
 C_T &:= 0.035 \\
 T &:= C_T \cdot h_n^{(3+4)} = 0.58 && \text{Fundamental period, Per 1113.4.5.2, Eq 3} \\
 C &:= \min\left(\frac{1}{15 \cdot \sqrt{T}}, 0.12\right) = 0.088 && \text{Numerical coefficient Per 1113.4.4 C factor} \\
 S &:= 1.5 && \text{Per 1113.4.6, assume soil profile=S3} \\
 CS &:= \min(C \cdot S, 0.14) = 0.131 && \text{Per 1113.4.4 C factor CS limit} \\
 V_{\text{seismicEW}} &:= Z \cdot K \cdot I \cdot CS \cdot W = 1.152 \times 10^5 && \text{lb}
 \end{aligned}$$

Lateral Load

Wind Load

Wind load in this calculation determined by using BOCA 1987

Basic on the Map [Figure 1112.3.2]

St.Louis has a basic wind speed in miles per hour=70 mph

Structural Height=43 ft

From Table 1112.3.3a, the effective velocity pressures for this building =

WindPressure := 13 psf

Wal Pressure Coefficient Cp:

From Table 1112.2a

Windward wall = 0.8

Leeward wall = -0.5 When ($0 < L/B < 1$)

= -0.3 When ($1 < L/B < 2$)

= -0.2 When ($4 < L/B$)

Side walls = 0.7

Importance Factor: I := 1.0

Floor Height:

Height3 := 14.5 ft

Height2 := 13.75 ft

Height1 := 13.75 ft

N-S:

$$\text{Height}_{\text{Total}} := \text{Height3} + \text{Height2} + \text{Height1} = 42 \quad \text{ft}$$

$$\text{Width3}_{\text{NS}} := 142 \quad \text{ft}$$

$$\text{Width2}_{\text{NS}} := 142$$

$$\text{Width1}_{\text{NS}} := 142$$

$$\frac{\text{Height}_{\text{Total}}}{\text{Width1}_{\text{NS}}} = 0.296 \quad < 2.5, \text{ Windward } C_p W = 0.8 \text{ Leeward } C_p L = 0.5$$

$$C_{pW} := 0.8$$

$$C_{pL} := 0.5$$

Roof Diaphragm:

$$\text{WindwardAreaRoof}_{\text{NS}} := \frac{\text{Height3}}{2} \cdot \text{Width3}_{\text{NS}} = 1.03 \times 10^3 \quad \text{sf}$$

$$\text{WindwardLoadRoof}_{\text{NS}} := \text{WindwardAreaRoof}_{\text{NS}} \cdot C_{pW} \cdot I^2 \cdot \text{WindPressure} = 1.07 \times 10^4 \quad \text{lb}$$

$$\text{LeewardAreaRoof}_{\text{NS}} := \frac{\text{Height3}}{2} \cdot \text{Width3}_{\text{NS}} = 1.03 \times 10^3 \quad \text{sf}$$

$$\text{LeewardLoadRoof}_{\text{NS}} := \text{LeewardAreaRoof}_{\text{NS}} \cdot C_{pL} \cdot I^2 \cdot \text{WindPressure} = 6.69 \times 10^3 \quad \text{lb}$$

$$\text{WindLoadRoof}_{\text{NS}} := \text{WindwardLoadRoof}_{\text{NS}} + \text{LeewardLoadRoof}_{\text{NS}} = 1.174 \times 10^4 \quad \text{lb}$$

3rd Floor Diaphragm:

$$\text{WindwardArea3rd}_{\text{NS}} := \left(\frac{\text{Height3}}{2} + \frac{\text{Height2}}{2} \right) \cdot \text{Width2}_{\text{NS}} = 2.006 \times 10^3 \quad \text{sf}$$

$$\text{WindwardLoad3rd}_{\text{NS}} := \text{WindwardArea3rd}_{\text{NS}} \cdot C_{pW} \cdot I^2 \cdot \text{WindPressure} = 2.09 \times 10^4 \quad \text{lb}$$

$$\text{LeewardArea3rd}_{\text{NS}} := \left(\frac{\text{Height3}}{2} + \frac{\text{Height2}}{2} \right) \cdot \text{Width2}_{\text{NS}} = 2.006 \times 10^3 \quad \text{sf}$$

$$\text{LeewardLoad3rd}_{\text{NS}} := \text{LeewardArea3rd}_{\text{NS}} \cdot C_{pL} \cdot I^2 \cdot \text{WindPressure} = 1.3 \times 10^4 \quad \text{lb}$$

$$\text{WindLoad3rd}_{\text{NS}} := \text{WindwardLoad3rd}_{\text{NS}} + \text{LeewardLoad3rd}_{\text{NS}} = 3.39 \times 10^4 \quad \text{lb}$$

2nd Floor Diaphragm:

$$\text{WindwardArea2nd}_{\text{NS}} := \left(\frac{\text{Height2}}{2} + \frac{\text{Height1}}{2} \right) \cdot \text{Width1}_{\text{NS}} = 1.952 \times 10^3 \quad \text{sf}$$

$$\text{WindwardLoad2nd}_{\text{NS}} := \text{WindwardArea2nd}_{\text{NS}} \cdot C_{p_w} \cdot I^2 \cdot \text{WindPressure} = 2.03 \times 10^4 \quad \text{lb}$$

$$\text{LeewardArea2nd}_{\text{NS}} := \left(\frac{\text{Height2}}{2} + \frac{\text{Height1}}{2} \right) \cdot \text{Width1}_{\text{NS}} = 1.952 \times 10^3 \quad \text{sf}$$

$$\text{LeewardLoad2nd}_{\text{NS}} := \text{LeewardArea2nd}_{\text{NS}} \cdot C_{p_L} \cdot I^2 \cdot \text{WindPressure} = 1.27 \times 10^4 \quad \text{lb}$$

$$\text{WindLoad2nd}_{\text{NS}} := \text{WindwardLoad2nd}_{\text{NS}} + \text{LeewardLoad2nd}_{\text{NS}} = 3.3 \times 10^4 \quad \text{lb}$$

BASE SHEAR FROM WIND

$$\text{WindLoadBase}_{\text{NS}} := \text{WindLoadRoof}_{\text{NS}} + \text{WindLoad3rd}_{\text{NS}} + \text{WindLoad2nd}_{\text{NS}} = 7.863 \times 10^4 \quad \text{lb}$$

E-W

$$\text{Height}_{\text{Total}} = 42 \quad \text{ft}$$

$$\text{Width}_{3\text{EW}} := 70 \quad \text{ft}$$

$$\text{Width}_{2\text{EW}} := 70 \quad \text{ft}$$

$$\text{Width}_{1\text{EW}} := 76 \quad \text{ft}$$

$$\frac{\text{Height}_{\text{Total}}}{\text{Width}_{3\text{EW}}} = 0.6 \quad < 2.5, \text{ Windward } C_{pW}=0.8 \text{ Leeward } C_{pL}=0.5$$

$$C_{pW} := 0.8$$

$$C_{pL} := 0.5$$

Roof Diaphragm:

$$\text{WindwardAreaRoof}_{\text{EW}} := \frac{\text{Height}_3}{2} \cdot \text{Width}_{3\text{EW}} = 507.5 \quad \text{sf}$$

$$\text{WindwardLoadRoof}_{\text{EW}} := \text{WindwardAreaRoof}_{\text{EW}} \cdot C_{pW} \cdot I^2 \cdot \text{WindPressure} = 5.28 \times 10^3 \quad \text{lb}$$

$$\text{LeewardAreaRoof}_{\text{EW}} := \frac{\text{Height}_3}{2} \cdot \text{Width}_{3\text{EW}} = 507.5 \quad \text{sf}$$

$$\text{LeewardLoadRoof}_{\text{EW}} := \text{LeewardAreaRoof}_{\text{EW}} \cdot C_{pL} \cdot I^2 \cdot \text{WindPressure} = 3.3 \times 10^3 \quad \text{lb}$$

$$\text{WindLoadRoof}_{\text{EW}} := \text{WindwardLoadRoof}_{\text{EW}} + \text{LeewardLoadRoof}_{\text{EW}} = 5.785 \times 10^3 \quad \text{lb}$$

3rd Floor Diaphragm:

$$\text{WindwardArea3rd}_{\text{EW}} := \left(\frac{\text{Height}_3}{2} + \frac{\text{Height}_2}{2} \right) \cdot \text{Width}_{2\text{EW}} = 988.75 \quad \text{sf}$$

$$\text{WindwardLoad3rd}_{\text{EW}} := \text{WindwardArea3rd}_{\text{EW}} \cdot C_{pW} \cdot I^2 \cdot \text{WindPressure} = 1.03 \times 10^4 \quad \text{lb}$$

$$\text{LeewardArea3rd}_{\text{EW}} := \left(\frac{\text{Height}_3}{2} + \frac{\text{Height}_2}{2} \right) \cdot \text{Width}_{2\text{EW}} = 988.75 \quad \text{sf}$$

$$\text{LeewardLoad3rd}_{\text{EW}} := \text{LeewardArea3rd}_{\text{EW}} \cdot C_{pL} \cdot I^2 \cdot \text{WindPressure} = 6.43 \times 10^3 \quad \text{lb}$$

$$\text{WindLoad3rd}_{\text{EW}} := \text{WindwardLoad3rd}_{\text{EW}} + \text{LeewardLoad3rd}_{\text{EW}} = 1.671 \times 10^4 \quad \text{lb}$$

2nd Floor Diaphragm:

$$\text{WindwardArea2nd}_{EW} := \left(\frac{\text{Height2}}{2} + \frac{\text{Height1}}{2} \right) \cdot \text{Width1}_{NS} = 1.952 \times 10^3 \quad \text{sf}$$

$$\text{WindwardLoad2nd}_{EW} := \text{WindwardArea2nd}_{NS} \cdot C_{pw} \cdot I^2 \cdot \text{WindPressure} = 2.03 \times 10^4 \quad \text{lb}$$

$$\text{LeewardArea2nd}_{EW} := \left(\frac{\text{Height2}}{2} + \frac{\text{Height1}}{2} \right) \cdot \text{Width1}_{NS} = 1.952 \times 10^3 \quad \text{sf}$$

$$\text{LeewardLoad2nd}_{EW} := \text{LeewardArea2nd}_{EW} \cdot C_{pL} \cdot I^2 \cdot \text{WindPressure} = 1.27 \times 10^4 \quad \text{lb}$$

$$\text{WindLoad2nd}_{EW} := \text{WindwardLoad2nd}_{EW} + \text{LeewardLoad2nd}_{EW} = 3.3 \times 10^4 \quad \text{lb}$$

BASE SHEAR FROM WIND

$$\text{WindLoadBase}_{EW} := \text{WindLoadRoof}_{EW} + \text{WindLoad3rd}_{EW} + \text{WindLoad2nd}_{EW} = 5.549 \times 10^4 \quad \text{lb}$$

Design Lateral Load

N-S(yy):

$$V_{\text{Wind}} := \text{WindLoadBase}_{\text{NS}} = 7.863 \times 10^4 \quad \text{lb}$$

$$V_{\text{Seismic}} := V_{\text{seismicNS}} = 1.228 \times 10^5 \quad \text{lb}$$

ControlCheck = "Seismic Control"

$$V_{\text{NS}} := \frac{V_{\text{Seismic}}}{1000} = 122.849 \quad \text{Kips}$$

E-W(xx):

$$V_{\text{Wind}} := \text{WindLoadBase}_{\text{EW}} = 5.549 \times 10^4 \quad \text{lb}$$

$$V_{\text{Seismic}} := V_{\text{seismicEW}} = 1.152 \times 10^5 \quad \text{lb}$$

ControlCheck = "Seismic Control"

$$V_{\text{EW}} := \frac{V_{\text{Seismic}}}{1000} = 115.219 \quad \text{Kips}$$

MOMENT FRAME DESIGN

Moment Frame Design Criteria

1. Moment-resisting Ordinary steel frames using $K=1.0$ shall have a frame conforming to the requirements for the AISC Spec listed in Appendix A
2. Drift limitation: Lateral Deflection(or drift) shall not exceed 0.005 times the story height

$$\text{Height3} := 14.5 \quad \text{ft}$$

$$\text{Height2} := 13.75 \quad \text{ft}$$

$$\text{Height1} := 13.75 \quad \text{ft}$$

Lateral Deflection Limit:

$$\text{Drift3} := 0.005 \cdot \text{Height3} \cdot 12 \cdot \frac{K}{I} = 0.87 \quad \text{inches}$$

$$\text{Drift2} := 0.005 \cdot \text{Height2} \cdot 12 \cdot \frac{K}{I} = 0.825 \quad \text{inches}$$

$$\text{Drift1} := 0.005 \cdot \text{Height1} \cdot 12 \cdot \frac{K}{I} = 0.825 \quad \text{inches}$$

Roof Diaphragm:

Flexible Diaphragm, force distributed basic on their tributary area(mass).

$$\text{Area1} := 1652 \quad \text{sf}$$

$$\text{Area2} := 5003 \quad \text{sf}$$

$$\text{Area3} := 3240 \quad \text{sf}$$

Total Lateral Force at Roof (E-W):

$$\text{TotalLateralForce} := 23.725 \quad \text{Kips}$$

Force distribution at the Roof:

$$F_{\text{Roof_MF1}} := \text{TotalLateralForce} \cdot \frac{\text{Area1}}{\text{Area1} + \text{Area2} + \text{Area3}} = 3.961 \quad \text{Kips}$$

$$F_{\text{Roof_MF2}} := \text{TotalLateralForce} \cdot \frac{\text{Area2}}{\text{Area1} + \text{Area2} + \text{Area3}} = 11.996 \quad \text{Kips}$$

$$F_{\text{Roof_MF3}} := \text{TotalLateralForce} \cdot \frac{\text{Area3}}{\text{Area1} + \text{Area2} + \text{Area3}} = 7.768 \quad \text{Kips}$$

Stiffness R:

Start with assuming the stiffness will be similar to each other at first trial,

Input the location and stiffness of each frames(include momentframe and braceframe) to Rigid Diaphragm distribution Spreadsheet:

ThirdFloor Diaphragm:

Moment Frame

MF1 := 19.28 kips

MF2 := 21.26 kips

MF3 := 21.46 kips

Similarly at Second Floor

Moment Frame

MF1 := 9.33 kips

MF2 := 10.29 kips

MF3 := 10.38 kips

Moment Frame Design:

MF1:

MF1

ROOF := 3.961 Kips

THIRD := 19.28 Kips

SECOND := 9.33 Kips

Roof Dead Load:

Dead Load 20 psf

Trib Width = 14.33 ft

$$20 \cdot \frac{14.333}{1000} = 0.287 \text{ klf}$$

Live Load 20 psf

Trib Width = 14.33 ft

$$20 \cdot \frac{14.333}{1000} = 0.287 \text{ klf}$$

Floor Dead Load:

Dead Load 90 psf

Trib Width = 14.33 ft

$$90 \cdot \frac{14.333}{1000} = 1.29 \text{ klf}$$

Live Load 50 psf

Trib Width = 14.33 ft

$$50 \cdot \frac{14.333}{1000} = 0.717 \text{ klf}$$

Drift Limit:

Drift1 = 0.825

Drift2 = 0.825

Drift3 = 0.87

Deflection1 := Drift1 = 0.825

Deflection2 := Deflection1 + Drift2 = 1.65

Deflection3 := Deflection2 + Drift3 = 2.52

By Iteration, the following Beam/Column Configuration satisfies the Drift/Strength requirement

Check real stiffness at Frame 1

1 Kips of Unit Load at Second Floor

$$E1_2 := 1 \text{ kips}$$

$$\Delta1_2 := 0.0212 \text{ in}$$

$$MF1_R := \frac{E1_2}{\Delta1_2} = 47.17 \frac{\text{Kips}}{\text{in}}$$

1 Kips of Unit Load at Third Floor

$$E1_3 := 1 \text{ kips}$$

$$\Delta1_3 := 0.036 \text{ in}$$

$$MF1_R := \frac{E1_3}{\Delta1_3} = 27.397 \frac{\text{Kips}}{\text{in}}$$

MF2:

MF2

ROOF := 11.996

THIRD := 21.26

SECOND := 10.29

Roof Dead Load:

Dead Load 20 psf

Trib Width = 19 ft

$$20 \cdot \frac{19}{1000} = 0.38 \text{ klf}$$

Live Load 20 psf

Trib Width = 19 ft

$$20 \cdot \frac{19}{1000} = 0.38 \text{ klf}$$

Floor Dead Load:

Dead Load 90 psf

Trib Width = 19 ft

$$90 \cdot \frac{19}{1000} = 1.71 \text{ klf}$$

Live Load 50 psf

Trib Width = 19 ft

$$50 \cdot \frac{19}{1000} = 0.95 \text{ klf}$$

Drift Limit:

Drift1 = 0.825

Drift2 = 0.825

Drift3 = 0.87

Deflection1 = 0.825

Deflection2 = 1.65

Deflection3 = 2.52

Check real stiffness at Frame 2

1 Kips of Unit Load at Second Floor

$$E2_2 := 1 \text{ kips}$$

$$\Delta2_2 := 0.016 \text{ in}$$

$$MF2_R := \frac{E2_2}{\Delta2_2} = 62.5 \frac{\text{Kips}}{\text{in}}$$

1 Kips of Unit Load at Third Floor

$$E2_3 := 1 \text{ kips}$$

$$\Delta2_3 := 0.027 \text{ in}$$

$$MF2_R := \frac{E2_3}{\Delta2_3} = 37.037 \frac{\text{Kips}}{\text{in}}$$

MF3:

MF3

ROOF := 7.77

THIRD := 21.46

SECOND := 10.38

Roof Dead Load:

$$\begin{aligned} \text{Dead Load} &= 20 \text{ psf} \\ \text{Trib Width} &= 14.65 \text{ ft} \\ 20 \cdot \frac{14.65}{1000} &= 0.293 \text{ klf} \end{aligned}$$

$$\begin{aligned} \text{Live Load} &= 20 \text{ psf} \\ \text{Trib Width} &= 14.65 \text{ ft} \\ 20 \cdot \frac{14.65}{1000} &= 0.293 \text{ klf} \end{aligned}$$

Floor Dead Load:

$$\begin{aligned} \text{Dead Load} &= 90 \text{ psf} \\ \text{Trib Width} &= 14.65 \text{ ft} \\ 90 \cdot \frac{14.65}{1000} &= 1.319 \text{ klf} \end{aligned}$$

$$\begin{aligned} \text{Live Load} &= 50 \text{ psf} \\ \text{Trib Width} &= 14.65 \text{ ft} \\ 50 \cdot \frac{14.65}{1000} &= 0.733 \text{ klf} \end{aligned}$$

Drift Limit:

$$\begin{aligned} \text{Drift1} &= 0.825 \\ \text{Drift2} &= 0.825 \\ \text{Drift3} &= 0.87 \end{aligned}$$

$$\begin{aligned} \text{Deflection1} &= 0.825 \text{ inches} \\ \text{Deflection2} &= 1.65 \text{ inches} \\ \text{Deflection3} &= 2.52 \text{ inches} \end{aligned}$$

Check real stiffness at Frame 3

1 Kips of Unit Load at Second Floor

$$E3_2 := 1 \text{ kips}$$

$$\Delta3_2 := 0.0162 \text{ in}$$

$$MF3_R := \frac{E3_2}{\Delta3_2} = 61.728 \frac{\text{Kips}}{\text{in}}$$

1 Kips of Unit Load at Third Floor

$$E3_3 := 1 \text{ kips}$$

$$\Delta3_3 := 0.028 \text{ in}$$

$$MF3_R := \frac{E3_3}{\Delta3_3} = 34.722 \frac{\text{Kips}}{\text{in}}$$

Project: PROJECT NAME

Job Number: 19051

Sheet: 1 of 2

Name: H.L

Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland

Date: 04/26/21

RIGID DIAPHRAGM DISTRIBUTION

LEVEL:

Second Floor

COMMENTS:

EQ LOAD IN ASD

EQUATIONS

$$M_t = F (y_{cf} - CR)$$

$$CR_x = \Sigma Ry_{datum} / \Sigma R$$

$$\%R_{\text{torsion}} = R_i x_{\text{cr}} / \sum (R(x_{\text{cr}})^2 + R(y_{\text{cr}})^2)$$

$$\%R = R_i / \sum R$$

$$x_{cr} = CR_x - x_{datum}$$

$$V_t = M^* \%R_{\text{torsion}}$$

$$V_{\text{direct}} = F \% R$$

$$\Delta = V_{\text{sum}}/R$$

E/W DIRECTION

ELEMENT	R	%R	Y _{datum}	R _{ydatum}	Y _{CR}	R _{yCR}	R(Y _{CR}) ²	%R _{torsion}	V _{direct}	V _t	V _{ta}	A _x V _{ta}	V _{total} + A _x V _{ta}	V _{total} - A _x V _{ta}	CRITICAL SUM DESIGN	CRITICAL SUM TORSION	Δ
	<i>k/in</i>		<i>ft</i>	<i>k/in-ft</i>	<i>ft</i>	<i>k/in-ft</i>	<i>k/in-ft²</i>		<i>k</i>	<i>k</i>	<i>k</i>	<i>k</i>	<i>k</i>	<i>k</i>	<i>k</i>	<i>k</i>	<i>in</i>
EW-1	47.20	0.27	67.00	3162.40	34.66	1636.03	5.67E+04	0.0013	8.22	0.05	0.14	0.14	8.41	8.13	8.41	8.41	0.18
EW-2	62.50	0.36	38.50	2406.25	6.16	385.11	2.37E+03	0.0003	10.89	0.01	0.03	0.03	10.93	10.87	10.93	10.93	0.17
EW-3	62.50	0.36	0.00	0.00	-32.34	-2021.14	6.54E+04	-0.0017	10.89	-0.06	-0.17	-0.17	11.00	10.66	11.00	10.66	0.17
		0.00	0.00	0.00	-32.34	0.00	0.00E+00	0.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
		 								

$$F_x = 30 \text{ k}$$

$$L_y = 67 \text{ ft}$$

$$CF_y = 34 \text{ ft}$$

$$CR_y = 32.34 \text{ ft}$$

$$A_x = 1.00$$

$$M_t = 35 \text{ k-ft}$$

$$M_{ta} = 101 \text{ k-ft} \quad \pm 5\%$$

$$\sum R(x_{cr})^2 + \sum R(y_{cr})^2 = 1.216E+06$$

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EQ LOAD IN ASD

$$\Delta = V_{\text{open}}/R$$

A _x =	1.00	
M _t =	38 k-ft	
M _{ta} =	208 k-ft	± 5%
ΣR(x _{cr}) ² +ΣR(y _{cr}) ² =	1.162E+06	

Input the location and **Update** stiffness of each frames(include momentframe and braceframe) to Rigid Diaphragm distribution third floor Spreadsheet:

ThirdFloor Diaphragm:

Moment Frame

MF1 := 17.33 kips

MF2 := 23.2 kips

MF3 := 21.47 kips

Similarly at Second Floor

Moment Frame

MF1 := 8.41 kips

MF2 := 10.93 kips

MF3 := 10.66 kips

Update Load at Each Frame:

MF1

ROOF := 3.961 Kips

THIRD := 17.33 Kips

SECOND := 8.41 Kips

MF2

ROOF := 11.996 Kips

THIRD := 23.2 Kips

SECOND := 10.93 Kips

MF3

ROOF := 7.77 Kips

THIRD := 21.47 Kips

SECOND := 10.66 Kips

Checked, Current design meet drift and strength requirement for updated force, see sample calculation

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in ²]	Iyy [in ⁴]	Izz [in ⁴]	J [in ⁴]
1	MF-BEA...	W21x68 ...	Beam	Wide Flan...	A36 Gr.36	Typical	20	64.7	1480	2.45
2	MF-BEA...	W21x57 ...	Beam	Wide Flan...	A36 Gr.36	Typical	16.7	30.6	1170	1.77
3	MF-BEA...	W16x45 ...	Beam	Wide Flan...	A36 Gr.36	Typical	13.3	32.8	586	1.11
4	BF-COLU...	W18x76 ...	Column	Wide Flan...	A36 Gr.36	Typical	22.3	152	1330	2.83

Node Loads and Enforced Displacements (BLC 3 : E)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (... Inactive [(k, k-ft), (in,...
1	N2	L	X	3.96 Active
2	N5	L	X	17.33 Active
3	N7	L	X	8.41 Active

Node Loads and Enforced Displacements (BLC 4 : Unit Load Second Floor)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (... Inactive [(k, k-ft), (in,...
1	N7	L	X	1 Active

Node Loads and Enforced Displacements (BLC 5 : Unit Load ThirdFloor)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (... Inactive [(k, k-ft), (in,...
1	N5	L	X	1 Active

Story Drift X-Service

	Story (El...		Story Dri...	Loc	LC	Drift Rati...	Loc	LC	2nd/1st...	Loc	LC
1	Diaph.: 1...	max	0.1289	0, 41	3	0.0761	0, 41	6	1.0121	0, 19	5
2		min	-0.1324	0, 41	6	0.0005	0, 19	7	1	0, 41	1
3	Diaph.: 2...	max	0.2887	0, 0	5	0.1827	0, 41	6	1.0177	0, 0	5
4		min	-0.296	0, 41	6	0.0015	0, 41	1	1	0, 41	7
5	Diaph.: 3...	max	0.7311	0, 0	5	0.4407	0, 0	6	1.0286	0, 0	2
6		min	-0.7404	0, 0	6	0.0018	0, 0	1	1	0, 0	7

Story Drift Z-Service

	Story (El...		Story Dri...	Loc	LC	Drift Rati...	Loc	LC	2nd/1st...	Loc	LC
1	Diaph.: 1...	max				NC			NC		
2		min				NC			NC		
3	Diaph.: 2...	max				NC			NC		
4		min				NC			NC		
5	Diaph.: 3...	max				NC			NC		
6		min				NC			NC		

Story Drift X-Strength

No Data to Print...											
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Story Drift Z-Strength

No Data to Print...											
---------------------	--	--	--	--	--	--	--	--	--	--	--

Member Distributed Loads (BLC 1 : DL)

	Member Label	Direction	Start Magnitud...	End Magnitude...	Start Location [...	End Location [(... Inactive [(k, k-f...
1	M4	Y	-0.287	-0.287	0	%100 Active
2	M5	Y	-0.287	-0.287	0	%100 Active
3	M8	Y	-1.29	-1.29	0	%100 Active
4	M9	Y	-1.29	-1.29	0	%100 Active
5	M6	Y	-1.29	-1.29	0	%100 Active
6	M7	Y	-1.29	-1.29	0	%100 Active

Member Distributed Loads (BLC 2 : LL)

	Member Label	Direction	Start Magnitud...	End Magnitude...	Start Location [...]	End Location [...]	Inactive [(k, k-f...
1	M4	Y	-0.287	-0.287	0	%100	Active
2	M5	Y	-0.287	-0.287	0	%100	Active
3	M8	Y	-0.717	-0.717	0	%100	Active
4	M9	Y	-0.717	-0.717	0	%100	Active
5	M6	Y	-0.717	-0.717	0	%100	Active
6	M7	Y	-0.717	-0.717	0	%100	Active

Maximum Member Section Forces

blic...	Me...		Axi...	Loc...	LC	y S...	Loc...	LC	z S...	Loc...	LC	Tor...	Loc...	LC	y-y...	Loc...	LC	z-z...	Loc...	LC
1	M1	max	67....	0	6	8.4...	14	3	0	42	8	0	42	8	0	42	8	149...	14	6
2		min	-0.7...	0	8	-10....	0	6	0	0	1	0	0	1	0	0	1	-11...	14	3
3	M2	max	108...	0	5	11.7...	14	3	0	42	8	0	42	8	0	42	8	176...	14	6
4		min	0.0...	27....	7	-13....	14	6	0	0	1	0	0	1	0	0	1	-16...	14	3
5	M3	max	70....	0	5	10....	14	5	0	42	8	0	42	8	0	42	8	109...	14	4
6		min	0.0...	27....	7	-7.8...	0	4	0	0	1	0	0	1	0	0	1	-15...	14	5
7	M4	max	0	19	8	7.333	0	6	0	19	8	0	19	8	0	19	8	32....	0	6
8		min	0	0	1	-7.2...	19	5	0	0	1	0	0	1	0	0	1	-12....	7.3...	5
9	M5	max	0	22	8	7.9...	0	6	0	22	8	0	22	8	0	22	8	36....	22	5
10		min	0	0	1	-7.955	22	5	0	0	1	0	0	1	0	0	1	-15....	12....	6
11	M6	max	0	19	8	32....	0	6	0	19	8	0	19	8	0	19	8	194...	0	6
12		min	0	0	1	-35....	19	5	0	0	1	0	0	1	0	0	1	-11...	0	3
13	M7	max	0	22	8	35....	0	6	0	22	8	0	22	8	0	22	8	200...	22	5
14		min	0	0	1	-33....	22	5	0	0	1	0	0	1	0	0	1	-96....	16....	6
15	M8	max	0	19	8	24....	0	6	0	19	8	0	19	8	0	19	8	117....	19	5
16		min	0	0	1	-25....	19	5	0	0	1	0	0	1	0	0	1	-44....	6.5...	5
17	M9	max	0	22	8	27....	0	6	0	22	8	0	22	8	0	22	8	130...	0	6
18		min	0	0	1	-26....	22	5	0	0	1	0	0	1	0	0	1	-54....	13....	6

Detail Report: M7

Load Combination: Envelope

Input Data:

Shape:	W21x68_ASD8th	I Node:	N11
Member Type:	Beam	J Node:	N12
Length (ft):	22	I Release:	Fixed
Material Type:	Hot Rolled Steel	J Release:	Fixed
Design Rule:	Typical	I Offset (in):	N/A
Number of Internal Sections:	97	J Offset (in):	N/A

Material Properties:

Material:	A36 Gr.36	Therm. Coeff. (1e ⁻⁵ F ⁻¹):	0.65	R _y :	1.5
E (ksi):	29000	Density (k/ft ³):	0.49	F _u (ksi):	58
G (ksi):	11154	F _y (ksi):	36	R _t :	1.2
Nu:	0.3				

Shape Properties:

d (in):	21.13	Area (in ²):	20	S _w (in ⁴):	59.864
b _f (in):	8.27	Z _{yy} (in ³):	24.4	r _T (in):	2.12
t _f (in):	0.685	Z _{zz} (in ³):	160	J (in ⁴):	2.45
t _w (in):	0.43	C _w (in ⁶):	6761.604	k _{det} (in):	0
I _{yy} (in ⁴):	64.7	W _{no} (in ²):	42.27	k _{des} (in):	1.438
I _{zz} (in ⁴):	1480				

Design Properties:

L _{b y-y} (ft):	22	K _{y-y} :	1	Max Defl Ratio:	L/2345
L _{b z-z} (ft):	22	K _{z-z} :	1	Max Defl Location:	14.208
comp top (ft):	22	y sway:	No	Span:	1
comp bot (ft):	22	z sway:	No		
C _b :	1.895	Function:	Lateral		
C _{m y-y} :	0.6	Seismic DR:	None		
C _{m z-z} :	0.85

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AISC Eighth Edition Column Design Wide Flange Member
Only

Project ID:	19051
Project Name:	ATC
Engineer:	HL
Date:	11/20/2020

Member # = MF1-B1

Shape W 21x68

Properties	Geometry	
	A	20 in. ²
	d	21.13 in.
	b _f	8.27 in.
	t _f	0.685 in.
	t _w	0.43 in.
	W(weight)	68 #/ft.
	r _T	2.12
	d/A _f	3.73

Compact Section Criteria	
b _f /2t _f	6
F _y	— ksi
d/t _w	49.1
F _y	— ksi
F _y	27.4 ksi

Elastic Properties					
X-X			Y-Y		
I _x	1480	in. ⁴	I _y	64.7	in. ⁴
S _x	140	in. ³	S _y	15.7	in. ³
r _x	8.6	in.	r _y	1.8	in.
Plastic Modulus					
Z _x	160	in. ³	Z _y	24.4	in. ³

Torsional Constant	
J	2.45 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Eighth Edition

Building info:

Steel Properties

Total Member Length =	22 ft.
Beam Spacing (w) =	12 ft.
Tributary Area to Col=	0 ft.^2
K=	1
Unbraced Length =	12 ft.

Loading

Dead Load (D)=	95 psf
Live Load(L)=	50 psf
Roof Live Load(Lr)=	0 psf
Snow Load(S)=	25 psf

Excentricity on Column

ex =	2 in.
ey =	0 in.

Deflection Criteria

Total Load	L/360=	0.73 in.
------------	--------	----------

Max Moment About Major Axis:

MD(Max Dead)=	827.64 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	827.64 kips-in
ML(Max Live)=	435.6 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	435.6 kips-in
M(Lr or S)(Max Live)=	217.8 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	217.8 kips-in
Mu=	2412 kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Shear

VD(Max Dead)=	12.54 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	12.54 kips
VL(Max Live)=	6.6 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	6.6 kips
V(Lr or S)(Max Live)=	5.016 kips
V(Lr or S)(Live Overwrite)=	0 kips
V(Lr or S)(Max Use)=	5.016 kips
Vu=	36 kips

Max Axial Load

PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0.1 kips
PD(Max Use)=	0.1 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0.1 kips
PL(Max Use)=	0.1 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0.1 kips
P(Lr or S)(Max Use)=	0.1 kips
P(E or W)=	1 kips
Pu=	1 kips

Solution Summary:

Try **W 21x68** <---- Change W shapes

Member	Major Axis Beam Bending	54%
	Beam Shear	28%
	Minor Axis Beam Bending	0%
	Axial	1%
	Deflection	34%
	Combined Bending and Axial	55%
Overall Check		55%

Column Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Column Height= 22 ft
K= 1
KL= 22 ft
KL/r= 146.7

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	126
if $\frac{KL}{r} \leq C_c$ then $F_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}]F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
if $\frac{KL}{r} \geq C_c$ then $F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2
Column in Tension	
$F_a = 0.6F_y$	Section 1.5.1.1

Fa = 6.9 ksi

Calculate Stresses in Column

Axial Stress: fa= Pu/A= 0.1 ksi Okay Increases by 1/3

Non-Composite Major Axis Beam Design AISC Eighth Edition

Unbraced length (Lb)= 12 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f/2t_f \leq 65/\sqrt{F_y}$	6	10.8	Good
$d/t_w \leq 412(1 - 2.33\frac{f_u}{F_y})/\sqrt{F_y}$	49.1	68.2	Good
$l_b \leq 76b_f/\sqrt{F_y}$	144	104.8	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d/A_f) F_y}$	144	148.9	Good
			Use Alternate Equation

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2/\sqrt{F_y} < b_f/2t_f$	6	8.7	Good
$b_f/2t_f \leq 95.0/\sqrt{F_y}$	6	15.8	Good
			Use Alternate Bending Stress Equation

Typical Bending Stress Equation: $F_{bx} = 0.66F_y$

Fbx= N/A ksi Increases by 1/3

Alternate Bending Stress Equation: $F_{bx} = F_y[0.733 - 0.0014(\frac{b_f}{t_f})\sqrt{F_y}]$

Fbx= 31.92 ksi Increases by 1/3

Shear Stress Equation: $F_v = 0.40F_y$

Fv= 14.4 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.75F_y$

Fby= 27 ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	17.2	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w*d) =$	4	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0.25	in.	Okay

Check Combined Stressed

$C_{mx} =$	1
$C_{my} =$	1

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - \frac{f_a}{F_{ex}}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - \frac{f_a}{F_{ey}}) F_{by}} =$	0.55	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.54	Less than 1.0, Okay

SAMPLE COLUMN CALC

Company : <Licensed Company>
 Designer : HLIN
 Job Number :
 Model Name :

4/26/2021
 4:45:40 PM
 Checked By : _____

Detail Report: M3

Load Combination: Envelope

Input Data:

Shape:	W18x76_ASD8th	I Node:	N9
Member Type:	Column	J Node:	N10
Length (ft):	42	I Release:	Fixed
Material Type:	Hot Rolled Steel	J Release:	Fixed
Design Rule:	Typical	I Offset (in):	N/A
Number of Internal Sections:	97	J Offset (in):	N/A

Material Properties:

Material:	A36 Gr.36	Therm. Coeff. (1e ⁻⁵ °F ⁻¹):	0.65	R _y :	1.5
E (ksi):	29000	Density (k/ft ³):	0.49	F _u (ksi):	58
G (ksi):	11154	F _y (ksi):	36	R _t :	1.2
Nu:	0.3				

Shape Properties:

d (in):	18.21	Area (in ²):	22.3	S _w (in ⁴):	90.723
b _f (in):	11.035	Z _{yy} (in ³):	42.2	r _T (in):	2.95
t _f (in):	0.68	Z _{zz} (in ³):	163	J (in ⁴):	2.83
t _w (in):	0.425	C _w (in ⁶):	11708.083	k _{det} (in):	0
I _{yy} (in ⁴):	152	W _{no} (in ²):	48.361	k _{des} (in):	1.375
I _{zz} (in ⁴):	1330				

Design Properties:

L _{b y-y} (ft):	42	K _{y-y} :	1	Max Defl Ratio:	L/0
L _{b z-z} (ft):	42	K _{z-z} :	1	Max Defl Location:	0
L _{comp top} (ft):	42	y sway:	No	Span:	N/A
L _{comp bot} (ft):	42	z sway:	No		
C _b :	1	Function:	Lateral		
C _{m y-y} :	0.6	Seismic DR:	None		
C _{m z-z} :	0.6

Diagrams:

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v3B>

AISC Eighth Edition Column Design Wide Flange Member Only		Project ID:	19051
			Project Name:	ATC
			Engineer:	HL
			Date:	11/20/2020

Member # =	MF1-C1	Shape	W 18x76
------------	--------	-------	---------

Properties	Geometry	
	A	22.3 in. ²
	d	18.21 in.
	b _f	11.035 in.
	t _f	0.68 in.
	t _w	0.425 in.
	W(weight)	76 #/ft.
	r _T	2.95
	d/A _f	2.43

Compact Section Criteria	
b _f /2t _f	8.1
F _y	64.2 ksi
d/t _w	42.8
F _y	— ksi
F _y	36 ksi

Elastic Properties					
X-X			Y-Y		
I _x	1330	in. ⁴	I _y	152	in. ⁴
S _x	146	in. ³	S _y	27.6	in. ³
r _x	7.73	in.	r _y	2.61	in.
Plastic Modulus					
Z _x	163	in. ³	Z _y	42.2	in. ³

Torsional Constant	
J	2.83 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Eighth Edition

Building info:

Steel Properties

Total Member Length =	14.5	ft.
Beam Spacing (w) =	0	ft.
Tributary Area to Col=	0	ft.^2
K=	1	
Unbraced Length =	14.5	ft.

Loading

Dead Load (D)=	80	psf
Live Load(L)=	60	psf
Roof Live Load(Lr)=	0	psf
Snow Load(S)=	25	psf

Excentricity on Column

ex =	2	in.
ey =	0	in.

Deflection Criteria

Total Load	L/360=	0.48	in.
------------	--------	------	-----

Max Moment About Major Axis:

MD(Max Dead)=	0.2	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0.2	kips-in
ML(Max Live)=	0.2	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0.2	kips-in
M(Lr or S)(Max Live)=	0.2	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0.2	kips-in
Mu=	1320	kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Shear

VD(Max Dead)=	0	kips
VD(Dead Overwrite)=	0	kips
VMD(Max Use)=	0	kips
VL(Max Live)=	0	kips
VL(Live Overwrite)=	0	kips
VL(Max Use)=	0	kips
V(Lr or S)(Max Live)=	0	kips
V(Lr or S)(Live Overwrite)=	0	kips
V(Lr or S)(Max Use)=	0	kips
Vu=	11	kips

Max Axial Load

PD(Max Dead)=	0	kips
PD(Dead Overwrite)=	0.1	kips
PD(Max Use)=	0.1	kips
PL(Max Live)=	0	kips
PL(Live Overwrite)=	0.1	kips
PL(Max Use)=	0.1	kips
P(Lr or S)(Max Live)=	0	kips
P(Lr or S)(Live Overwrite)=	0.1	kips
P(Lr or S)(Max Use)=	0.1	kips
P(E or W)=	71	kips
Pu=	71	kips

Solution Summary:

Try **W 18x76** <---- Change W shapes

Member	Major Axis Beam Bending	28%
	Beam Shear	10%
	Minor Axis Beam Bending	0%
	Axial	19%
	Deflection	0%
	Combined Bending and Axial	48%
Overall Check		48%

Column Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Column Height= 14.5 ft
K= 1
KL= 14.5 ft
KL/r= 66.7

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	126
if $\frac{KL}{r} \leq C_c$ then $F_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}]F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
if $\frac{KL}{r} \geq C_c$ then $F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2
Column in Tension	
$F_a = 0.6F_y$	Section 1.5.1.1

Fa = 16.8 ksi

Calculate Stresses in Column

Axial Stress: fa= Pu/A= 3.2 ksi Okay Increases by 1/3

Non-Composite Major Axis Beam Design AISC Eighth Edition

Unbraced length (Lb)= 14.5 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f/2t_f \leq 65/\sqrt{F_y}$	8.1	10.8	Good
$d/t_w \leq 412(1 - 2.33\frac{f_a}{F_y})/\sqrt{F_y}$	42.8	54.4	Good
$l_b \leq 76b_f/\sqrt{F_y}$	174	139.8	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d/A_f) F_y}$	174	228.9	Good
			Use Alternate Equation

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2/\sqrt{F_y} < b_f/2t_f$	8.1	8.7	Good
$b_f/2t_f \leq 95.0/\sqrt{F_y}$	8.1	15.8	Good
			Use Alternate Bending Stress Equation

Typical Bending Stress Equation: $F_{bx} = 0.66F_y$

Fbx= N/A ksi Increases by 1/3

Alternate Bending Stress Equation: $F_{bx} = F_y[0.733 - 0.0014(\frac{b_f}{t_f})\sqrt{F_y}]$

Fbx= 31.8 ksi Increases by 1/3

Shear Stress Equation: $F_v = 0.40F_y$

Fv= 14.4 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.75F_y$

Fby= 27 ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	9	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	1.4	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5wL^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

$C_{mx} =$	1
$C_{my} =$	1

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - \frac{f_a}{F_{ex}}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - \frac{f_a}{F_{ey}}) F_{by}} =$	0.48	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.43	Less than 1.0, Okay

BRACED FRAME DESIGN

Brace Frame Design Criteria

1. All members in braced frames shall be designed for **1.25** times the lateral force determined from BOCA 1987.

1113.9.5 Braced frames: In Seismic Zones 3 and 4, and for buildings having an importance factor I greater than 1.0 and located in Seismic Zone 2, all members in braced frames shall be designed for 1.25 times the force determined in accordance with Section 1113.4. Steel members in braced frames shall be limited to those grades listed in Section 1113.9.3.3. Reinforced concrete members in braced frames shall be provided with the transverse confinement reinforcement required in Section 1113.9.3.3.

2. Drift limitation: Lateral Deflection(or drift) shall not exceed 0.005 times the story height per 1113.7 Displacement calculated from application shall be multiplied by I/K

Height3 := 14.5 ft

Height2 := 13.75 ft

Height1 := 13.75 ft

Lateral Deflection Limit:

$$\text{Drift1} := 0.005 \cdot \text{Height1} \cdot 12 \cdot \frac{K}{I} = 0.66 \text{ inches}$$

$$\text{Drift2} := 0.005 \cdot \text{Height2} \cdot 12 \cdot \frac{K}{I} = 0.66 \text{ inches}$$

$$\text{Drift3} := 0.005 \cdot \text{Height3} \cdot 12 \cdot \frac{K}{I} = 0.696 \text{ inches}$$

Roof Diaphragm:

Flexible Diaphragm, force distributed basic on their tributary area(mass).

Area1 := 3310 sf

Area2 := 3210 sf

Area3 := 3550 sf

Total Lateral Force at Roof (N-S):

TotalLateralForce := 38.4 Kips (From Vertical Distribution)

Force at OCBF 1 Roof:

$$F_{\text{Roof_OCBF1}} := \text{TotalLateralForce} \cdot \frac{\text{Area1}}{\text{Area1} + \text{Area2} + \text{Area3}} = 12.622 \text{ Kips}$$

$$F_{\text{Roof_OCBF2}} := \text{TotalLateralForce} \cdot \frac{\text{Area2}}{\text{Area1} + \text{Area2} + \text{Area3}} = 12.241 \text{ Kips}$$

$$F_{\text{Roof_OCBF3}} := \text{TotalLateralForce} \cdot \frac{\text{Area3}}{\text{Area1} + \text{Area2} + \text{Area3}} = 13.537 \text{ Kips} \quad \text{Control}$$

In N-S Direction,

3 Brace Frames are identical and they will have similar stiffness.

Stiffness R:

$$\frac{29.5}{0.108} = 273.148 \quad \frac{\text{kips}}{\text{in}}$$

Since all three braces frames are very similar, we will design with the worst case

Lateral Load at different diaphragm: (Note 25% force increase are required per 1113.9.5)

Roof:	$\text{MaxRoof_OCBF} \cdot 1.25 = 16.922$	Kips	
Third Floor:	$36.6 \cdot 1.25 = 45.75$	Kips	<-----From Spreadsheet
Second Floor:	$17.84 \cdot 1.25 = 22.3$	Kips	<-----From Spreadsheet

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in ²]	Iyy [in ⁴]	Izz [in ⁴]	J [in ⁸]
1	BF-BEAM...	W18x50 ...	Beam	Wide Flan...	A36 Gr.36	Typical	14.7	40.1	800	1.24
2	BF-BEAM...	W18x55 ...	Beam	Wide Flan...	A36 Gr.36	Typical	16.2	44.9	890	1.66
3	BF-BEAM...	W16x50 ...	Beam	Wide Flan...	A36 Gr.36	Typical	14.7	37.2	659	1.52
4	BF-COLU...	W14x68 ...	Column	Wide Flan...	A36 Gr.36	Typical	20	121	723	3.02
5	BF-BRAC...	STS4x4x...	VBrace	Tube	A500 Gr...	Typical	4.36	10.521	9.58	15.669
6	BF-BRAC...	STS3.5x3...	VBrace	Tube	A500 Gr...	Typical	3.73	6.812	6.09	10.12
7	BF-BRAC...	STS5x5x...	VBrace	Tube	A500 Gr...	Typical	5.61	21.553	20.1	32.187

Node Loads and Enforced Displacements (BLC 3 : E)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (... Inactive [(k, k-ft), (in,...
1	N2	L	X	10.611 Active
2	N5	L	X	28.4 Active
3	N7	L	X	14 Active

Story Drift X-Service

	Story (El...		Story Dri...	Loc	LC	Drift Rati...	Loc	LC	2nd/1st...	Loc	LC
1	N2 (42 ft)	max	0.1507	0, 0	5	0.0866	0, 0	5	1.0005	0, 0	5
2		min	-0.1493	0, 0	4	0	0, 0	1	1	0, 0	1
3	N5 (27.5 ft)	max	0.3164	0, 0	3	0.1966	0, 0	6	1.0008	0, 0	6
4		min	-0.3185	0, 0	6	0	0, 0	1	1	0, 0	1
5	N7 (14 ft)	max	0.3259	0, 0	3	0.1945	0, 0	6	1.0007	0, 0	6
6		min	-0.3268	0, 0	6	0	0, 0	1	1	0, 0	1

Story Drift Z-Service

	Story (El...		Story Dri...	Loc	LC	Drift Rati...	Loc	LC	2nd/1st...	Loc	LC
1	N2 (42 ft)	max				NC			NC		
2		min				NC			NC		
3	N5 (27.5 ft)	max				NC			NC		
4		min				NC			NC		
5	N7 (14 ft)	max				NC			NC		
6		min				NC			NC		

Story Drift X-Strength

No Data to Print...											
---------------------	--	--	--	--	--	--	--	--	--	--	--

Story Drift Z-Strength

No Data to Print...											
---------------------	--	--	--	--	--	--	--	--	--	--	--

Member Distributed Loads (BLC 1 : DL)

	Member Label	Direction	Start Magnitud...	End Magnitude...	Start Location [...	End Location [(...	Inactive [(k, k-f...
1	M3	Y	-0.2	-0.2	0	%100	Active
2	M4	Y	-0.9	-0.9	0	%100	Active
3	M5	Y	-0.9	-0.9	0	%100	Active

Member Distributed Loads (BLC 2 : LL)

	Member Label	Direction	Start Magnitud...	End Magnitude...	Start Location [...	End Location [(...	Inactive [(k, k-f...
1	M5	Y	-0.5	-0.5	0	%100	Active
2	M4	Y	-0.5	-0.5	0	%100	Active
3	M3	Y	-0.2	-0.2	0	%100	Active

Maximum Member Section Forces

	Me...		Axi...	Loc...	LC	y S...	Loc...	LC	z S...	Loc...	LC	Tor...	Loc...	LC	y-y...	Loc...	LC	z-z...	Loc...	LC
1	M1	max	66....	0	6	0.3...	27....	3	0	42	6	0	42	6	0	42	6	4.3...	27....	6
2		min	-3.0...	27....	3	-0.403	14	6	0	0	1	0	0	1	0	0	1	-4.2...	27....	3

Maximum Member Section Forces (Continued)

	Me...		Axi...		Loc...		LC	y S...		Loc...		LC	z S...		Loc...		LC	Tor...		Loc...		LC	y-y...		Loc...		LC	z-z...		Loc...		LC
3	M2	max	64....	0	5	0.3...	27....	5	0	42	6	0	42	6	0	42	6	3.172	27....	4												
4		min	-1.3...	27....	4	-0.2...	14	4	0	0	1	0	0	1	0	0	1	-3.283	27....	5												
5	M3	max	10....	14.65	3	3.8...	14....	5	0	29.3	6	0	29.3	6	0	29.3	6	9.7...	14.65	5												
6		min	-10....	0	6	-3.9...	14.65	5	0	0	1	0	0	1	0	0	1	-8.3...	6.1...	6												
7	M4	max	12....	29.3	3	21....	0	5	0	29.3	6	0	29.3	6	0	29.3	6	0	29.3	6												
8		min	-14....	0	6	-21....	29.3	5	0	0	1	0	0	1	0	0	1	-15...	14.65	5												
9	M5	max	52....	14.65	5	12....	14....	6	0	29.3	6	0	29.3	6	0	29.3	6	34....	14.65	6												
10		min	-52....	14.65	6	-13....	14.65	6	0	0	1	0	0	1	0	0	1	-24....	5.799	5												
11	M6	max	13....	0	6	0.1...	0	6	0	20....	6	0	20....	6	0	20....	6	0	20....	6												
12		min	-4.5...	20....	3	-0.1...	20....	3	0	0	1	0	0	1	0	0	1	-0.5...	10....	6												
13	M7	max	13....	20....	5	0.0...	0	3	0	20....	6	0	20....	6	0	20....	6	0	20....	6												
14		min	-4.6...	0	4	-0.0...	20....	5	0	0	1	0	0	1	0	0	1	-0.51	10....	3												
15	M10	max	51....	0	6	0.1...	0	5	0	20....	6	0	20....	6	0	20....	6	0	20....	6												
16		min	-25....	20....	3	-0.1...	20....	2	0	0	1	0	0	1	0	0	1	-0.7...	10....	5												
17	M11	max	53....	20....	5	0.1...	0	6	0	20....	6	0	20....	6	0	20....	6	0	20....	6												
18		min	-27....	0	4	-0.1...	20....	3	0	0	1	0	0	1	0	0	1	-0.754	10....	6												
19	M12	max	24....	19....	3	0.11...	0	6	0	19....	6	0	19....	6	0	19....	6	0	19....	6												
20		min	-31....	0	6	-0.1...	19....	4	0	0	1	0	0	1	0	0	1	-0.5...	9.9...	3												
21	M13	max	22....	0	4	0.11...	0	5	0	19....	6	0	19....	6	0	19....	6	0	19....	6												
22		min	-29....	19....	5	-0.1...	19....	3	0	0	1	0	0	1	0	0	1	-0.5...	9.9...	5												

charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v3B>

AISC Eighth Edition Column Design Wide Flange Member Only

Project ID:	19051
Project Name:	ATC
Engineer:	HL
Date:	11/20/2020

Member # = **BF1-B2**

Shape **W 18x55**

Properties	Geometry	
	A	16.2 <i>in.^2</i>
	d	18.11 <i>in.</i>
	b_f	7.53 <i>in.</i>
	t_f	0.63 <i>in.</i>
	t_w	0.39 <i>in.</i>
	W(weight)	55 <i>#/ft.</i>
	r_T	1.95
	d/A_f	3.82

Compact Section Criteria	
b_f/2t_f	6
F_y	— <i>ksi</i>
d/t_w	46.4
F_y	— <i>ksi</i>
F_y	30.6 <i>ksi</i>

Elastic Properties					
X-X			Y-Y		
I_x	890	<i>in.^4</i>	I_y	44.9	<i>in.^4</i>
S_x	98.3	<i>in.^3</i>	S_y	11.9	<i>in.^3</i>
r_x	7.41	<i>in.</i>	r_y	1.67	<i>in.</i>
Plastic Modulus					
Z_x	112	<i>in.^3</i>	Z_y	18.5	<i>in.^3</i>

Torsional Constant	
J	1.66 <i>in.^4</i>

Warping Constant	
C_w	— <i>in.^6</i>

Material Property	
E	29000 <i>ksi</i>
F_y	36 <i>ksi</i>

Non-Composite Beam/Column Design AISC Eighth Edition

Building info:

Steel Properties

Total Member Length =	29.33	ft.
Beam Spacing (w) =	0	ft.
Tributary Area to Col=	0	ft.^2
K=	1	
Unbraced Length =	14.5	ft.

Loading

Dead Load (D)=	80	psf
Live Load(L)=	60	psf
Roof Live Load(Lr)=	0	psf
Snow Load(S)=	25	psf

Excentricity on Column

ex =	2	in.
ey =	0	in.

Deflection Criteria

Total Load	L/360=	0.98	in.
------------	--------	------	-----

Max Moment About Major Axis:

MD(Max Dead)=	0.2	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0.2	kips-in
ML(Max Live)=	0.2	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0.2	kips-in
M(Lr or S)(Max Live)=	0.2	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0.2	kips-in
Mu=	1884	kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Shear

VD(Max Dead)=	0	kips
VD(Dead Overwrite)=	0	kips
VMD(Max Use)=	0	kips
VL(Max Live)=	0	kips
VL(Live Overwrite)=	0	kips
VL(Max Use)=	0	kips
V(Lr or S)(Max Live)=	0	kips
V(Lr or S)(Live Overwrite)=	0	kips
V(Lr or S)(Max Use)=	0	kips
Vu=	22	kips

Max Axial Load

PD(Max Dead)=	0	kips
PD(Dead Overwrite)=	0.1	kips
PD(Max Use)=	0.1	kips
PL(Max Live)=	0	kips
PL(Live Overwrite)=	0.1	kips
PL(Max Use)=	0.1	kips
P(Lr or S)(Max Live)=	0	kips
P(Lr or S)(Live Overwrite)=	0.1	kips
P(Lr or S)(Max Use)=	0.1	kips
P(E or W)=	14.5	kips
Pu=	14.5	kips

Solution Summary:

Try **W 18x55** <---- Change W shapes

Member	Major Axis Beam Bending	60%
	Beam Shear	22%
	Minor Axis Beam Bending	0%
	Axial	26%
	Deflection	0%
	Combined Bending and Axial	87%
Overall Check		87%

Column Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Column Height= 29.33 ft
K= 1
KL= 29.33 ft
KL/r= 210.8

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	126
if $\frac{KL}{r} \leq C_c$ then $F_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 15.1.3.1
if $\frac{KL}{r} \geq C_c$ then $F_a = \frac{12\pi^2 E}{23 (\frac{KL}{r})^2}$	Section 15.1.3.2
Column in Tension	
$F_a = 0.6F_y$	Section 15.1.1

Fa = 3.4 ksi

Calculate Stresses in Column

Axial Stress: fa= Pu/A= 0.9 ksi Okay Increases by 1/3

Non-Composite Major Axis Beam Design AISC Eighth Edition

Unbraced length (Lb)= 14.5 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 65 / \sqrt{F_y}$	6	10.8	Good
$d / t_w \leq 412 (1 - 2.33 \frac{F_a}{F_y}) / \sqrt{F_y}$	46.4	64.7	Good
$l_b / \leq 76 b_f / \sqrt{F_y}$	174	95.4	No Good, Use Alternate EQN
$l_b / \leq \frac{20,000}{(d / A_f) F_y}$	174	145.5	No Good, Use Alternate EQN
			Use Alternate Equation

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	6	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	6	15.8	Good
			Use Alternate Bending Stress Equation

Typical Bending Stress Equation: $F_{bx} = 0.66F_y$

Fbx= N/A ksi Increases by 1/3

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

Fbx= 31.92 ksi Increases by 1/3

Shear Stress Equation: $F_v = 0.40F_y$

Fv= 14.4 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.75F_y$

Fby= 27 ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	19.2	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w*d) =$	3.1	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5wL^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

$C_{mx} =$	1
$C_{my} =$	1

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{C_{mx}f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{C_{my}f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.87	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.64	Less than 1.0, Okay

AISC Eighth Edition Column Design Wide Flange Member Only

Project ID:	19051
Project Name:	ATC
Engineer:	HL
Date:	11/20/2020

Member # = **BF1-C1**

Shape **W 14x61**

Properties	Geometry	
	A	17.9 <i>in.²</i>
	d	13.89 <i>in.</i>
	b_f	9.995 <i>in.</i>
	t_f	0.645 <i>in.</i>
	t_w	0.375 <i>in.</i>
	W(weight)	61 <i>#/ft.</i>
	r_T	2.7
	d/A_f	2.15

Compact Section Criteria	
b_f/2t_f	7.7
F_y	— <i>ksi</i>
d/t_w	37
F_y	— <i>ksi</i>
F_y	48.1 <i>ksi</i>

Elastic Properties			
X-X		Y-Y	
I_x	640 <i>in.⁴</i>	I_y	107 <i>in.⁴</i>
S_x	92.2 <i>in.³</i>	S_y	21.5 <i>in.³</i>
r_x	5.98 <i>in.</i>	r_y	2.45 <i>in.</i>
Plastic Modulus			
Z_x	102 <i>in.³</i>	Z_y	32.8 <i>in.³</i>

Torsional Constant	
J	2.2 <i>in.⁴</i>

Warping Constant	
C_w	— <i>in.⁶</i>

Material Property	
E	29000 <i>ksi</i>
F_y	36 <i>ksi</i>

Non-Composite Beam/Column Design AISC Eighth Edition

Building info:

Steel Properties

Total Member Length =	14.5	ft.
Beam Spacing (w) =	0	ft.
Tributary Area to Col=	0	ft.^2
K=	1	
Unbraced Length =	14.5	ft.

Loading

Dead Load (D)=	80	psf
Live Load(L)=	60	psf
Roof Live Load(Lr)=	0	psf
Snow Load(S)=	25	psf

Excentricity on Column

ex =	2	in.
ey =	0	in.

Deflection Criteria

Total Load	L/360=	0.48	in.
------------	--------	------	-----

Max Moment About Major Axis:

MD(Max Dead)=	0.2	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0.2	kips-in
ML(Max Live)=	0.2	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0.2	kips-in
M(Lr or S)(Max Live)=	0.2	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0.2	kips-in
Mu=	54	kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Shear

VD(Max Dead)=	0	kips
VD(Dead Overwrite)=	0	kips
VMD(Max Use)=	0	kips
VL(Max Live)=	0	kips
VL(Live Overwrite)=	0	kips
VL(Max Use)=	0	kips
V(Lr or S)(Max Live)=	0	kips
V(Lr or S)(Live Overwrite)=	0	kips
V(Lr or S)(Max Use)=	0	kips
Vu=	13	kips

Max Axial Load

PD(Max Dead)=	0	kips
PD(Dead Overwrite)=	0.1	kips
PD(Max Use)=	0.1	kips
PL(Max Live)=	0	kips
PL(Live Overwrite)=	0.1	kips
PL(Max Use)=	0.1	kips
P(Lr or S)(Max Live)=	0	kips
P(Lr or S)(Live Overwrite)=	0.1	kips
P(Lr or S)(Max Use)=	0.1	kips
P(E or W)=	80	kips
Pu=	80	kips

Solution Summary:

Try **W 14x61** <---- Change W shapes

Member	Major Axis Beam Bending	2%
	Beam Shear	17%
	Minor Axis Beam Bending	0%
	Axial	28%
	Deflection	0%
	Combined Bending and Axial	30%
Overall Check		30%

Column Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Column Height= 14.5 ft
K= 1
KL= 14.5 ft
KL/r= 71

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	126
if $\frac{KL}{r} \leq C_c$ then $F_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}]F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
if $\frac{KL}{r} \geq C_c$ then $F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2
Column in Tension	
$F_a = 0.6F_y$	Section 1.5.1.1

Fa = 16.3 ksi

Calculate Stresses in Column

Axial Stress: fa= Pu/A= 4.5 ksi Okay Increases by 1/3

Non-Composite Major Axis Beam Design AISC Eighth Edition

Unbraced length (Lb)= 14.5 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f/2t_f \leq 65/\sqrt{F_y}$	7.7	10.8	Good
$d/t_w \leq 412(1 - 2.33\frac{f_a}{F_y})/\sqrt{F_y}$	37	48.7	Good
$l_b \leq 76b_f/\sqrt{F_y}$	174	126.6	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d/A_f) F_y}$	174	257.9	Good
			Use Alternate Equation

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2/\sqrt{F_y} < b_f/2t_f$	7.7	8.7	Good
$b_f/2t_f \leq 95.0/\sqrt{F_y}$	7.7	15.8	Good
			Use Alternate Bending Stress Equation

Typical Bending Stress Equation: $F_{bx} = 0.66F_y$

Fbx= N/A ksi Increases by 1/3

Alternate Bending Stress Equation: $F_{bx} = F_y[0.733 - 0.0014(\frac{b_f}{t_f})\sqrt{F_y}]$

Fbx= 31.92 ksi Increases by 1/3

Shear Stress Equation: $F_v = 0.40F_y$

Fv= 14.4 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.75F_y$

Fby= 27 ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	0.6	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w*d) =$	2.5	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

$C_{mx} =$	1
$C_{my} =$	1

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - \frac{f_a}{F_{ex}}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - \frac{f_a}{F_{ey}}) F_{by}} =$	0.3	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.23	Less than 1.0, Okay

AISC Eighth Edition HSS Brace Member Only

Project ID:	19051
Project Name:	ATC
Engineer:	H.L
Date:	12/12/2020

Member # = BF Bottom Brace

Shape ST S 5x5x0.3125

Properties	Geometry	
	A	5.61 in. ²
	d	5 in.
	b	5 in.
	t	0.3125 in.
	W(weight)	19.08 #/ft.

Compact Section Criteria	
b/t	16
h/t	16

Material Property	
E	29000 ksi
Fy	46 ksi

Elastic Properties					
X-X			Y-Y		
I _x	20.1	in. ⁴	I _y	20.1	in. ⁴
S _x	8.02	in. ³	S _y	8.02	in. ³
r _x	1.89	in.	r _y	1.89	in.

Non-Composite Beam/Column Design AISC Eighth Edition

Building info:

Steel Properties

Column Height =	20.83	ft
Tributary Area to Col=	0	ft^2
K factor =	1	

Loading

Dead Load (D)=	80	psf
Live Load(L)=	60	psf
Roof Live Load(Lr)=	0	psf
Snow Load(S)=	25	psf

Excentricity on Column

ex =	0	in
ey =	0	in

Max Moment About Major Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Axial Load

PD(Max Dead)=	0	kips
PD(Dead Overwrite)=	0	kips
PD(Max Use)=	0	kips
PL(Max Live)=	0	kips
PL(Live Overwrite)=	0	kips
PL(Max Use)=	0	kips
P(Lr or S)(Max Live)=	0	kips
P(Lr or S)(Live Overwrite)=	0	kips
P(Lr or S)(Max Use)=	0	kips
P(E or W)=	53.5	kips
Pu=	53.5	kips

Solution Summary:Try **ST S 5x5x0.3125** Change Tube shapes

Member	Major Axis Beam Bending	0%
	Minor Axis Beam Bending	0%
	Axial	84%
	Combined Bending and Axial	84%
Overall Check		84%

Column Design AISC Eighth Edition

Column Height= 20.83 ft
 K= 1
 KL= 20.83 ft
 KL/r= 132.3

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	111
$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2
Column in Tension	
$F_a = 0.6F_y$	section 1.5.1.1

Fa = 11.305 ksi Note if Seismic/Wind loads occurs, 1/3 allowable stress will be used

Calculate Stresses in Column

Axial Stress: fa= Pu/A 9.5 ksi Column Okay

Non-Composite Major Axis Beam Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Checks for Typical Major Bending Stress Equation		
Equation	Variable Value	Limit Value
$b/t \leq 238 / \sqrt{F_y}$	16	35.1
Major Axis Bending Equation: $F_{bx} = 0.6F_y$		Fbx= 36.708 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.6F_y$ Fby = 36.708 ksi

Calculate Stresses in Beam

Major Bending Stress: fbx= Mux/Sx=	0	ksi	Column Okay
Minor Bending Stress: fby= Muy/Sy=	0	ksi	Column Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.84	Less than 1.0, Column Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.34	Less than 1.0, Column Okay

AISC Eighth Edition ORDINARY CONCENTRIC
BRACED FRAME CONNECTION DESIGN

Project ID:	19051
Project Name:	ATC
Engineer:	HL
Date:	11/22/2020

Braced Frame # 2nd Floor

Brace Size: ST S 5x5x0.3125

Brace Properties	
A	5.61 in ²
t	0.3125 in
B	5 in
F _y	36 ksi
Ultimate Axial Load applied to Brace=	56.5 kips

Weld Properties	
D	5/16"

Gusset Plate Properties	
tp	0.5 in
F _{yp}	36 ksi
E	29000 ksi
K	0.7
L _p	6 in
L	7 in
r	0.144 in
KL/r	29.2
C _c	126

Brace Yielding	$P_a = 0.6F_y A_{net} =$	114.4	kips	Okay
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Required Weld Length	$L_w = \frac{P_a}{0.928D * 4} =$	3	in
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Gusset Plate Yielding	$P_a = 0.6F_{yp} A_p =$	75.6	kips	Okay
-----------------------	-------------------------	------	------	------

Gusset Plate Buckling	$P_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}]F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3} A_p =$	70	kips	Okay
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2018 IBC Risk Category II Seismic Design Category C

This section presents representative structural calculations for Building 3 (3-story education facility in St. Louis, Missouri) Design Case B. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v3A.

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DESIGN CRITERIA

Search Information

Address: St. Louis, MO, USA
Coordinates: 38.62700249999999, -90.19940419999999
Elevation: 462 ft
Timestamp: 2020-08-24T21:13:20.719Z
Hazard Type: Wind

ASCE 7-16

MRI 10-Year 74 mph
MRI 25-Year 81 mph
MRI 50-Year 86 mph
MRI 100-Year 93 mph
Risk Category I 101 mph
Risk Category II 106 mph
Risk Category III 114 mph
Risk Category IV 118 mph

ASCE 7-10

MRI 10-Year 76 mph
MRI 25-Year 84 mph
MRI 50-Year 90 mph
MRI 100-Year 96 mph
Risk Category I 105 mph
Risk Category II 115 mph
Risk Category III-IV 120 mph

ASCE 7-05

ASCE 7-05 Wind Speed 90 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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Search Information

Address: St. Louis, MO, USA

Coordinates: 38.62700249999999, -90.19940419999999

Elevation: 462 ft

Timestamp: 2020-08-24T21:24:12.535Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D

MCE_R Horizontal Response Spectrum

Design Horizontal Response Spectrum

Basic Parameters

Name	Value	Description
S _S	0.453	MCE _R ground motion (period=0.2s)
S ₁	0.16	MCE _R ground motion (period=1.0s)
S _{MS}	0.651	Site-modified spectral acceleration value
S _M	0.365	Site-modified spectral acceleration value
S _{D0.2}	0.434	Numeric seismic design value at 0.2s SA
S _{D1.0}	0.244	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F _a	1.438	Site amplification factor at 0.2s
F _v	2.279	Site amplification factor at 1.0s
CR _S	0.909	Coefficient of risk (0.2s)
CR ₁	0.863	Coefficient of risk (1.0s)
PGA	0.261	MCE _G peak ground acceleration
F _{PGA}	1.339	Site amplification factor at PGA
PGA _M	0.349	Site modified peak ground acceleration
T _L	12	Long-period transition period (s)
SsRT	0.453	Probabilistic risk-targeted ground motion (0.2s)

SsUH	0.498	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.16	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.186	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey [Seismic Design Web Services](#).

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Search Information

Address: St. Louis, MO, USA
Coordinates: 38.62700249999999, -90.19940419999999
Elevation: 462 ft
Timestamp: 2020-08-24T21:23:22.173Z
Hazard Type: Snow

ASCE 7-16

Ground Snow Load 20 lb/sqft

ASCE 7-10

Ground Snow Load 20 lb/sqft

ASCE 7-05

Ground Snow Load 20 lb/sqft

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer.

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DESIGN CRITERIA

VERTICAL LOADS

AREA	DESIGN DEAD LOAD	LIVE LOAD (1)	CONCENTRATED LOADS
CLASSROOM AND OFFICE	75 PSF	50 PSF	1000#
CORRIDORS ABOVE 1ST FLOOR	75 PSF	80 PSF	1000#
ROOF	20 PSF	20 PSF	-

(1) LIVE LOADS EXCEPT SNOW LOADS ARE REDUCED PER IBC SECTION 1607.11.

SNOW

$P_g = 20$ PSF = GROUND SNOW LOAD

$P_f = 0.7 C_e C_t I_s P_g$ = FLAT ROOF SNOW LOAD

$I_s = 1.0$ $C_e = 1.0$, $C_t = 1.0$. C_s = VARIES

VIBRATION DESIGN CRITERIA:

VIBRATION CONSIDERATIONS WERE DESIGNED PER THE RECOMMENDATIONS IN THE GUIDELINES FOR DESIGN AND CONSTRUCTION OF COMPUTER EQUIPMENT RESIDENCES. FOR CLASSROOM FLOORS A VIBRATION VELOCITY LIMIT OF 8000 MICRO INCHES PER SECOND WAS EVALUATED AT A WALKING SPEED OF 75 STEPS PER MINUTE.

LATERAL FORCES

LATERAL FORCES ARE TRANSMITTED BY DIAPHRAGM ACTION OF ROOF AND FLOORS TO BRACED FRAME/MOMENT FRAME. LOADS ARE THEN TRANSFERRED TO FOUNDATION BY BRACED FRAME/MOMENT FRAME ACTION WHERE ULTIMATE DISPLACEMENT IS RESISTED BY PASSIVE PRESSURE OF EARTH AND/OR SLIDING FRICTION. OVERTURNING IS RESISTED BY DEAD LOAD OF THE STRUCTURE.

WIND

THE BUILDING MEETS THE CRITERIA TO USE THE "ENCLOSED, PARTIALLY ENCLOSED, AND OPEN BUILDING OF ALL HEIGHTS PROCEDURE" PER ASCE 7-16.

- EXPOSURE CATEGORY = B
- BASIC WIND SPEED, (3 SEC. GUST), $V_{ULT} = 106$ MPH; $V_{ASD} = 80$ MPH
- RISK CATEGORY PER IBC TABLE 1604.5 = II
- TOPOGRAPHIC FACTOR $K_{ZT} = 1.0$
- INTERNAL PRESSURE COEFFICIENT (ENCLOSED) = ± 0.18
- BASE SHEAR $V = 103$ KIPS AT BRACE FRAMES, 49.6 KIPS AT MOMENT FRAME

SEISMIC: (ASCE 7-16) $V = C_s W$

WHERE $C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$; WITH

$C_s \text{ MINIMUM} = 0.044 S_{DS} I_e \geq 0.01$

OR

$C_s \text{ MINIMUM} = \frac{0.5 S_1}{\frac{R}{I_e}} \text{ FOR } S_1 > 0.6g$

$C_s \text{ MAXIMUM} = \frac{S_{D1}}{T \left(\frac{R}{I_e}\right)} \text{ FOR } T \leq T_L$

OR

$C_s \text{ MAXIMUM} = \frac{S_{D1} T_L}{T^2 \left(\frac{R}{I_e}\right)} \text{ FOR } T > T_L$

SEISMIC IMPORTANCE FACTOR, $I_e = 1.0$

RISK CATEGORY OF BUILDING PER IBC TABLE 1604.5 = II

SPECTRAL RESPONSE ACCELERATIONS $S_s = 0.453$ & $S_1 = 0.16$

SITE CLASS PER TABLE 20.3-1 = D

DESIGN SPECTRAL RESPONSE ACCELERATIONS $S_{DS} = 0.434$ & $S_{D1} = 0.243$

SEISMIC DESIGN CATEGORY = C

W = EFFECTIVE SEISMIC WEIGHT OF BUILDING = 2140 KIPS

ANALYSIS PROCEDURE USED = EQUIVALENT LATERAL FORCE PROCEDURE/MODAL RESPONSE SPECTRUM ANALYSIS

RESPONSE MODIFICATION FACTOR PER TABLE 12.2-1, $R = 8.0$ AT SPECIAL STEEL MOMENT FRAMES, $R = 6.0$ AT SPECIAL CONCENTRICALLY STEEL BRACED FRAMES

$C_s = 0.037$ AT MOMENT FRAMES, $C_s = 0.072$ AT BRACED FRAMES

DESIGN BASE SHEAR $V = 154.92$ KIPS AT BRACED FRAMES, 80.00 KIPS AT MOMENT FRAMES

FOUNDATION DESIGN CRITERIA

SOIL BEARING PRESSURE: 4000 PSF (ASSUMED)*

ACTIVE PRESSURE - RESTRAINED: 50 PCF +14H SEISMIC SURCHARGE (ASSUMED)

ACTIVE PRESSURE - UNRESTRAINED: 35 PCF +6H SEISMIC SURCHARGE (ASSUMED)

PASSIVE RESISTANCE: 200 PCF (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

COEFFICIENT OF FRICTION: .35 (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

*1/3 INCREASE ALLOWED FOR SEISMIC OR WIND LOADING

ITEM	DESIGN f'_c (PSI) (AT 28 DAYS U.N.O.)
SLABS ON GRADE - UNO	4000
FOUNDATIONS - UNO	3000
STEM WALLS AND OTHER WALLS - UNO	4000
ELEVATED DECKS, TOPPING SLABS, AND SLABS ON METAL	5000

GRAVITY DESIGN

NOTE: BENTLEY RAM STRUCTURAL WAS USED AS DESIGN AIDS

ROOF BEAM DESIGN

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Floor Map

02/18/21 12:47:46

Steel Code: AISC 360-10 LRFD

Floor Type: Roof

Beam Numbers

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Beam Design

02/18/21 12:47:46

Steel Code: AISC 360-10 LRFD

Floor Type: Roof**Beam Number = 138****SPAN INFORMATION (ft): I-End (68.75,2.67) J-End (68.75,32.00)**

Beam Size (Optimum) = W16X26

Fy = 50.0 ksi

Total Beam Length (ft) = 29.33

Mp (kip-ft) = 184.17

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.190	0.000	---	Snow	0.000
	29.333	0.190	0.000			0.000
2	0.000	0.026	0.000	---	NonR	0.000
	29.333	0.026	0.000			0.000

SHEAR (Ultimate): Max Vu (1.4DL) = 4.44 kips 0.90Vn = 105.97 kips**MOMENTS (Ultimate):**

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.4DL	32.5	14.7	0.0	1.00	0.90	165.75
Controlling		1.4DL	32.5	14.7	0.0	1.00	0.90	165.75

REACTIONS (kips):

	Left	Right
DL reaction	3.17	3.17
Max +total reaction (factored)	4.44	4.44

DEFLECTIONS:

					Ratio
Dead load (in)	at 14.67 ft =	-0.412	L/D = 853 > 240		0.28
Live load (in)	at 14.67 ft =	-0.000			
Net Total load (in)	at 14.67 ft =	-0.412	L/D = 853 > 240		0.28

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FLOOR BEAM DESIGN

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Floor Map

11/18/20 09:27:36

Steel Code: AISC 360-10 LRFD

Floor Type: LEVEL 2

Beam Numbers

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Beam Design

02/11/21 15:54:03

Steel Code: AISC 360-10 LRFD

Floor Type: LEVEL 2**Beam Number = 135****SPAN INFORMATION (ft): I-End (59.25,2.67) J-End (59.25,32.00)**

Beam Size (Optimum) = W16X26

Fy = 50.0 ksi

Total Beam Length (ft) = 29.33

COMPOSITE PROPERTIES (Not Shored):

		Left	Right
Deck Label		3W+2.5	3W+2.5
Concrete thickness (in)		2.50	2.50
Unit weight concrete (pcf)		150.00	150.00
f _c (ksi)		4.00	4.00
Decking Orientation		perpendicular	perpendicular
Decking type		ASC 3W	ASC 3W
beff (in)	=	88.00	Y bar(in) = 17.42
M _{nf} (kip-ft)	=	406.66	M _n (kip-ft) = 280.02
C (kips)	=	103.38	PNA (in) = 11.99
I _{eff} (in ⁴)	=	770.41	I _{tr} (in ⁴) = 1205.71
Stud length (in)	=	4.50	Stud diam (in) = 0.75
Stud Capacity (kips)	Q _n = 17.2	R _g = 1.00	R _p = 0.60
# of studs:	Full = 52	Partial = 12	Actual = 12
Number of Stud Rows = 1	Percent of Full Composite Action = 26.92		

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.731	0.487	0.487	12.3%	Red	0.146	0.195
	29.333	0.731	0.487	0.487			0.146	0.195
2	0.000	0.026	0.026	0.000	---	NonR	0.000	0.000
	29.333	0.026	0.026	0.000			0.000	0.000

SHEAR (Ultimate): Max V_u (1.2DL+1.6LL) = 26.80 kips 0.90V_n = 105.97 kips**MOMENTS (Ultimate):**

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	99.8	14.7	0.0	1.00	0.90	165.75
	Init DL	1.4DL	77.3	14.7	---	---		
	Max +	1.2DL+1.6LL	196.5	14.7	---	---	0.90	252.02
Controlling		1.2DL+1.6LL	196.5	14.7	---	---	0.90	252.02

REACTIONS (kips):

	Left	Right
Initial reaction	10.39	10.39
DL reaction	11.11	11.11
Max +LL reaction	8.42	8.42
Max +total reaction (factored)	26.80	26.80

DEFLECTIONS: (Camber = 3/4)

			Ratio
Initial load (in)	at 14.67 ft =	-0.980	L/D = 359 > 200 0.56
Live load (in)	at 14.67 ft =	-0.428	L/D = 823 > 360 0.44

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Beam Design

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Steel Code: AISC 360-10 LRFD

Post Comp load (in)	at 14.67 ft = -0.610	L/D = 577 > 240	0.42
Net Total load (in)	at 14.67 ft = -0.840	L/D = 419 > 240	0.57

VIBRATION TO A TYPICAL BAY

FloorVibe V3.00, (C)2006-2018 by Structural Engineers, Inc.
www.floorvibe.com
Licensee: PCS Structural Solutions, Tacoma, Washington

SHEET OF
DATE: 6/25/2020
PAGE: 1 BY: HL

Project ID: ATC-NIST Bldg Comparison
Project # : 19051
Bay ID : Typ Bay

VIBRATION ANALYSIS:

Activity: Walking
Occupancy Category: Electronic Office
Evaluation Criterion: Walking
References: Murray, T.M., Allen, D.E., Ungar, E.E, Davis, D.B.
"Vibrations of Steel-Framed Structural Systems
Due To Human Activity", AISC Design Guide #11 2nd Ed, 2016
Murray, T.M., and Davis, B.,
"Vibration of Steel Joist-Concrete Floor Systems",
SJI Technical Digest No. 5, 2014
Constant Force, $P_o = 65.1b$
Modal Damping Ratio, $\beta = 0.025$
Acceleration Limit, $a_o/g \times 100\% = 0.50 \%$
Beams are not continuous
Girders are not continuous at columns

EVALUATION: Combined mode $a_p/g = 0.514 \%$ > 0.50%
This peak acceleration MAY BE ACCEPTABLE,
ENGINEERING JUDGEMENT IS REQUIRED.

PARAMETER SUMMARY:

Parameter	Beam Panel	Left Girder Panel	Right Girder Panel	System
Member	W16X40	W18X50	W18X50	
Uniform Load, plf	663.3	1307.0	1003.5	
Effective I, in ⁴	1924.0	2790.3	2507.2	
Frequency, Hz	8.27	6.52	7.05	5.12
Panel Width, ft	26.75	50.67	50.67	
Panel Weight, k	51.0	104.8	106.1	84.2

FRAMING:

Girder Span = 30.00 ft
Beam Spans:
Left = 9.15 ft
Center = 28.75 ft
Right = 0.00 ft
Girders/Walls:
Left -W18X50
Right-W18X50
Beam -W16X40
3 spaces at 120.00 in
Floor Width =120.00 ft
Floor Length= 76.00 ft
Concrete: dc = 5.50 in
f'c = 3.00 ksi
wt. = 145.0 pcf
Deck Height = 3.00 in

Loading: Dead = 4.00 psf
Live = 8.00 psf
Collateral = 0.00 psf

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Note: Floors with a frequency below 3 Hz are not recommended

COLUMN DESIGN

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Floor Map

11/18/20 09:27:36

Steel Code: AISC 360-10 LRFD

Floor Type: LEVEL 2

Beam Numbers

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Column Design

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02/11/21 16:03:27

Steel Code: AISC 360-10 LRFD

Story level Level 2, Column Line 6-C, Column # 31

Fy (ksi) = 50.00

Column Size

= W10X33

Orientation (deg.) = 90.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	13.75	13.75
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	7.36	6.48
Bottom _____	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	72.01	31.85	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 137.37	0.90Pnx (kip) = 390.25	Pu/0.90Pnx = 0.352
	0.90Pny (kip) = 257.68	Pu/0.90Pny = 0.533
	0.90Pn (kip) = 257.68	Pu/0.90Pn = 0.533

DEMAND/CAPACITY LIMIT FOR STRENGTH : 1.000**CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 10:**

	Dead	Live	Roof
Axial (kip) _____	72.01	28.24	0.00
Moments Top Mx (kip-ft) _____	-2.32	-1.20	0.00
My (kip-ft) _____	-0.61	-1.28	0.00
Bot Mx (kip-ft) _____	0.00	0.00	0.00
My (kip-ft) _____	0.00	0.00	0.00

Single curvature about X-Axis

Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 131.59	0.90*Pn (kip) = 257.68
Mux (kip-ft) = -4.71	0.90*Mnx (kip-ft) = 145.50
Muy (kip-ft) = -2.78	0.90*Mny (kip-ft) = 52.50
Rm = 1.00	
Cbx = 1.67	
Cmx = 0.60	Cmy = 0.60
Pex (kip) = 1797.74	Pey (kip) = 384.78
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.511

RAM Structural System

Gravity Column Design

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

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Steel Code: AISC 360-10 LRFD

$$\text{Eq H1-1a: } 0.511 + 0.029 + 0.047 = 0.587$$

LATERAL DESIGN

SEISMIC FORCE

IBC 2018 RISK Category II Education Building

From Spreadsheet,

N-S Direction, SCBF used as force resist element, $R=6$, $\Omega=2$, $C_s=0.072$

E-W Direction, SMF used as force resist element, $R=8$, $\Omega=3$, $C_s=C_{sMax}=0.054$

N-S:

$$I_{ns} := 1.0$$

$$R_{ns} := 6$$

From Table 12.2-1

$$\Omega_{ns} := 2.0$$

$$C_{d_{ns}} := 5$$

$$h_1 := 13.75$$

$$h_2 := 27.5 - 13.75 = 13.75$$

$$h_3 := 14.5$$

Allowable Story Drift: Δa_{ns}

$$\Delta a_{ns} := 0.025 \cdot h_{sx}$$

from Table 12.12-1

Max deflection(Drift):

from 12.8-15

Roof:

$$h_{sx} := h_1 \cdot 12$$

$$\Delta a_{ns} := 0.025 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ns} = 4.125$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ns}}{C_{d_{ns}}} = 0.825 \text{ in}$$

Third Floor:

$$h_{sx} := h_2 \cdot 12$$

$$\Delta a_{ns} := 0.025 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ns} = 4.125$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ns}}{C_{d_{ns}}} = 0.825 \text{ in}$$

Second Floor:

$$h_{sx} := h_3 \cdot 12$$

$$\Delta a_{ns} := 0.025 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ns} = 4.35$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ns}}{C_{d_{ns}}} = 0.87 \text{ in}$$

E-W:

From Table 12.2-1

$I_{ew} := 1.0$	$h_{sx1} := 13.75$	Height1 := 13.75 ft
$R_{ew} := 8$	$h_{sx2} := 27.5 - 13.75 = 13.75$	Height2 := 13.75 ft
$\Omega_{ew} := 3$	$h_{sx3} := 14.5$	Height3 := 14.5 ft
$Cd_{ew} := 5.5$		

Allowable Story Drift: Δa_{ns}

$$\Delta a_{ew} := 0.025 \cdot h_{sx} \quad \text{from Table 12.12-1}$$

Max deflection(Drift):

from 12.8-15

Roof:

$$\begin{aligned} h_{sx} &:= h_1 \cdot 12 \\ \Delta a_{ew} &:= 0.025 \cdot h_{sx} \\ \delta_x &:= \Delta a_{ew} = 4.125 \\ \delta_{xe} &:= \delta_x \cdot \frac{I_{ew}}{Cd_{ew}} = 0.75 \text{ in} \end{aligned}$$

Third Floor:

$$\begin{aligned} h_{sx} &:= h_2 \cdot 12 \\ \Delta a_{ew} &:= 0.025 \cdot h_{sx} \\ \delta_x &:= \Delta a_{ew} = 4.125 \\ \delta_{xe} &:= \delta_x \cdot \frac{I_{ew}}{Cd_{ew}} = 0.75 \text{ in} \end{aligned}$$

Second Floor:

$$\begin{aligned} h_{sx} &:= h_3 \cdot 12 \\ \Delta a_{ew} &:= 0.025 \cdot h_{sx} \\ \delta_x &:= \Delta a_{ew} = 4.35 \\ \delta_{xe} &:= \delta_x \cdot \frac{I_{ew}}{Cd_{ew}} = 0.791 \text{ in} \end{aligned}$$

By inspection, we notice there will be a torsional irregularity of Type 1a ($\delta_{avg} > 1.2(\text{Max}\delta)$) in Y direction
See next page for Torsional Irregularity Calc

$$\delta_{max} := 0.0868$$

$$\delta_{avg} := 0.0649$$

$$A_x := \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 = 1.242$$

When $A_x=1.0$ 5% of dimension of structure will be applied on both direction

Now in Y direction:

$$0.05 \cdot A_x = 0.062$$

New accidental moment 6.2% will be apply to design in Y direction

$$160 \cdot 0.062 = 9.92 \text{ ft}$$

Check Seismic Weight:

$$DL_w := 20 \text{ psf}$$

Diaphragm 1:

Floor Dead Load	$DL_1 := 80 \text{ psf}$	(Partional load = 10psf included as Seismi Weight)
Floor Area	$FloorA_1 := 9860 \text{ sf}$	
Building Perimeter	$P_1 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$	
Wal Height	$H_1 := \frac{(Height2 + Height1)}{2} = 13.75 \text{ ft}$	
	$W_1 := (DL_1 \cdot FloorA_1 + P_1 \cdot H_1 \cdot DL_w) = 9.267 \times 10^5 \text{ lb}$	

Diaphragm 2:

Floor Dead Load	$DL_2 := 80 \text{ psf}$	(Partional load = 10psf included as Seismi Weight)
Floor Area	$FloorA_2 := 10100 \text{ sf}$	
Building Perimeter	$P_2 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$	
Wal Height	$H_2 := \frac{(Height2 + Height3)}{2} = 14.125 \text{ ft}$	
	$W_2 := DL_2 \cdot FloorA_2 + P_2 \cdot H_2 \cdot DL_w = 9.497 \times 10^5 \text{ lb}$	

Diaphragm 3(ROOF):

Floor Dead Load	$DL_3 := 20 \text{ psf}$	
Floor Area	$FloorA_3 := 9560 \text{ sf}$	
Building Perimeter	$P_3 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$	
Wal Height	$H_3 := \frac{(Height3)}{2} = 7.25 \text{ ft}$	
	$W_3 := DL_3 \cdot FloorA_3 + P_3 \cdot H_3 \cdot DL_w = 2.639 \times 10^5 \text{ lb}$	
	$W := W_1 + W_2 + W_3 = 2.14 \times 10^6 \text{ lb}$

RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

02/11/21 16:10:51

Loads and Applied Forces

LOAD CASE: EQ

Seismic ASCE 7-16 Equivalent Lateral Force

Importance Factor: 1.00 TL: 12.00 s

Site Class D: Stiff Soil, Default

Ss: 0.453 g S1: 0.160 g

Fa: 1.438 Fv: 2.280

SDs: 0.434 g

SD1: 0.243 g

Use Specified Seismic Design Category: C

Provisions for: Force

Ground Level: Base

NOTE:
100% LATERAL LOAD ON ONE
DIRECTION +30% OTHER
DIRECTION WERE CONSIDERED IN
DESIGN BUT DID NOT PRINT.

Dir	Eccent	R	Ta Equation		Building Period-T				
X	+ And -	8.0	Std,Ct=0.028,x=0.80		Calculated				
Y	+ And -	6.0	Std,Ct=0.030,x=0.75		Calculated				
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
X	0.559	1.457	1.506	0.814	0.054	0.037	0.019	0.037	1.157
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
Y	0.496	1.457	0.388	0.388	0.072	0.104	0.019	0.072	1.000

Total Building Weight (kips) = 2141.01

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	19.25	0.00	70.05	38.26
Level 3	1	27.50	42.24	0.00	72.11	40.86
Level 2	1	13.75	18.49	0.00	66.05	41.87

APPLIED STORY FORCES

Type: EQ_ASCE716_X+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	19.25	0.00
Level 3	27.50	42.24	0.00
Level 2	13.75	18.49	0.00
		79.98	0.00

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RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Loads and Applied Forces

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APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	19.25	0.00	70.05	31.08
Level 3	1	27.50	42.24	0.00	72.11	33.02
Level 2	1	13.75	18.49	0.00	66.05	34.03

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	19.25	0.00
Level 3	27.50	42.24	0.00
Level 2	13.75	18.49	0.00
		<hr/> 79.98	<hr/> 0.00

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	0.00	34.49	79.95	34.67
Level 3	1	27.50	0.00	80.94	82.01	36.94
Level 2	1	13.75	0.00	39.49	75.95	37.95

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	0.00	34.49
Level 3	27.50	0.00	80.94
Level 2	13.75	0.00	39.49
		<hr/> 0.00	<hr/> 154.92

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	0.00	34.49	60.15	34.67
Level 3	1	27.50	0.00	80.94	62.21	36.94

RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Loads and Applied Forces

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Level 2	1	13.75	0.00	39.49	56.15	37.95
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APPLIED STORY FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	0.00	34.49
Level 3	27.50	0.00	80.94
Level 2	13.75	0.00	39.49
		0.00	154.92

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LOAD CASE: Winds

ASCE 7-16

Exposure: B

Basic Wind Speed (mph): 106.0

Apply Directionality Factor, $K_d = 0.85$ Use Topography Factor, $K_{zt} = 1.00$ Ground Elevation Factor, $K_e = 1.00$

Use Calculated Frequency for X-Dir.

Use Calculated Frequency for Y-Dir.

Gust Factor for Flexible Structures, G : Use Calculated G for X-Dir.Gust Factor for Rigid Structures, G : Use Calculated G for Y-Dir.

Damping Ratio for Flexible Structures = 0.01

Mean Roof Height (ft): Top Story Height + Parapet = 42.17

Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 0.664 Structure is Flexible

Y-Direction: Natural Frequency = 2.577 Structure is Rigid

 C_p Windward = 0.80 q Leeward (q_h) = 18.88 psf $G C_{pn}$ (Parapet): Windward = 1.50 Leeward = -1.00

Height ft	K_z	K_{zt}	q_z psf	Gust Factor G		C_p Leeward		Pressure (psf)	
				X	Y	X	Y	X	Y
42.17	0.772	1.000	18.879	1.013	0.820	-0.289	-0.500	20.828	20.122
27.50	0.683	1.000	16.709	1.001	0.820	-0.298	-0.500	19.018	18.698
13.75	0.575	1.000	14.052	1.001	0.820	-0.298	-0.500	16.890	16.956
0.00	0.575	1.000	14.052	1.001	0.820	-0.298	-0.500	16.890	16.956

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_X

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	10.65	0.00	68.75	34.92
Level 3	1	27.50	20.20	0.00	68.75	36.50
Level 2	1	13.75	18.57	0.00	68.75	38.17

APPLIED STORY FORCES

Type: Wind_ASCE716_1_X

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	10.65	0.00
Level 3	27.50	20.20	0.00
Level 2	13.75	18.57	0.00
		<hr/> 49.42	<hr/> 0.00

RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

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APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	0.00	22.98	68.75	34.92
Level 3	1	27.50	0.00	42.29	68.75	38.17
Level 2	1	13.75	0.00	37.82	68.75	38.17

APPLIED STORY FORCES

Type: Wind_ASCE716_1_Y

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	0.00	22.98
Level 3	27.50	0.00	42.29
Level 2	13.75	0.00	37.82
		<hr/> 0.00	<hr/> 103.10

COMPARE WIND/SEISMIC LOAD, WIND DOES NOT CONTROL

RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

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Steel Code: AISC360-16 LRFD

CRITERIA:

Rigid End Zones:	Ignore Effects
Member Force Output:	At Face of Joint
P-Delta: Yes	Scale Factor (DL): 1.20 Scale Factor (LL): 1.00
	Scale Factor (Roof): 1.00 Scale Factor (Snow): 1.00

Ground Level: Base

Mesh Criteria :

Max. Distance Between Nodes on Mesh Line (ft) : 4.00

Merge Node Tolerance (in) : 0.0150

Geometry Tolerance (in) : 0.0100

Walls Out-of-plane Stiffness Not Included in Analysis.

Sign considered for Dynamic Load Case Results.

Rigid Links Included at Fixed Beam-to-Wall Locations

Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

NOTE:
MOMENT FRAME CHECK AND
BRACED FRAME CHECK ARE BOTH
INCLUDED. SEE NEXT A FEW PAGES
FOR INDICATION.

LOAD COMBINATION CRITERIA:

Live Load factor f1 (0.5 or 1.0)	0.500
Sds (for Ev)	0.434
RhoX	1.000
RhoY	1.000

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
E1	EQ	EQ_ASCE716_X_+E_F
E2	EQ	EQ_ASCE716_X_-E_F
E3	EQ	EQ_ASCE716_Y_+E_F
E4	EQ	EQ_ASCE716_Y_-E_F
E5	EQ	EQ_ASCE716_X_+E_0.3Y_+E_F
E6	EQ	EQ_ASCE716_X_+E_-0.3Y_+E_F
E7	EQ	EQ_ASCE716_-X_+E_0.3Y_+E_F
E8	EQ	EQ_ASCE716_-X_+E_-0.3Y_+E_F
E9	EQ	EQ_ASCE716_0.3X_+E_Y_+E_F
E10	EQ	EQ_ASCE716_0.3X_+E_-Y_+E_F
E11	EQ	EQ_ASCE716_-0.3X_+E_Y_+E_F
E12	EQ	EQ_ASCE716_-0.3X_+E_-Y_+E_F
E13	EQ	EQ_ASCE716_X_+E_0.3Y_-E_F
E14	EQ	EQ_ASCE716_X_+E_-0.3Y_-E_F
E15	EQ	EQ_ASCE716_-X_+E_0.3Y_-E_F
E16	EQ	EQ_ASCE716_-X_+E_-0.3Y_-E_F
E17	EQ	EQ_ASCE716_0.3X_+E_Y_-E_F
E18	EQ	EQ_ASCE716_0.3X_+E_-Y_-E_F
E19	EQ	EQ_ASCE716_-0.3X_+E_Y_-E_F
E20	EQ	EQ_ASCE716_-0.3X_+E_-Y_-E_F
E21	EQ	EQ_ASCE716_X_-E_0.3Y_+E_F
E22	EQ	EQ_ASCE716_X_-E_-0.3Y_+E_F

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RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

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Steel Code: AISC360-16 LRFD

E23	EQ	EQ_ASCE716_-X_-E_0.3Y_+E_F
E24	EQ	EQ_ASCE716_-X_-E_-0.3Y_+E_F
E25	EQ	EQ_ASCE716_0.3X_-E_Y_+E_F
E26	EQ	EQ_ASCE716_0.3X_-E_-Y_+E_F
E27	EQ	EQ_ASCE716_-0.3X_-E_Y_+E_F
E28	EQ	EQ_ASCE716_-0.3X_-E_-Y_+E_F
E29	EQ	EQ_ASCE716_X_-E_0.3Y_-E_F
E30	EQ	EQ_ASCE716_X_-E_-0.3Y_-E_F
E31	EQ	EQ_ASCE716_-X_-E_0.3Y_-E_F
E32	EQ	EQ_ASCE716_-X_-E_-0.3Y_-E_F
E33	EQ	EQ_ASCE716_0.3X_-E_Y_-E_F
E34	EQ	EQ_ASCE716_0.3X_-E_-Y_-E_F
E35	EQ	EQ_ASCE716_-0.3X_-E_Y_-E_F
E36	EQ	EQ_ASCE716_-0.3X_-E_-Y_-E_F
W1	Winds	Wind_ASCE716_1_X
W2	Winds	Wind_ASCE716_1_Y
W3	Winds	Wind_ASCE716_2_X+E
W4	Winds	Wind_ASCE716_2_X-E
W5	Winds	Wind_ASCE716_2_Y+E
W6	Winds	Wind_ASCE716_2_Y-E
W7	Winds	Wind_ASCE716_3_X+Y
W8	Winds	Wind_ASCE716_3_X-Y
W9	Winds	Wind_ASCE716_4_X+Y_CW
W10	Winds	Wind_ASCE716_4_X+Y_CCW
W11	Winds	Wind_ASCE716_4_X-Y_CW
W12	Winds	Wind_ASCE716_4_X-Y_CCW

LOAD COMBINATIONS: IBC 2018 / ASCE 7-16 LRFD

1	*	1.400 D
2	*	1.200 D + 1.600 Lp
3	*	1.200 D + 1.600 Ln
4	*	1.200 D + 0.500 Lp + 1.000 W1
5	*	1.200 D + 0.500 Lp + 1.000 W2
6	*	1.200 D + 0.500 Lp + 1.000 W3
7	*	1.200 D + 0.500 Lp + 1.000 W4
8	*	1.200 D + 0.500 Lp + 1.000 W5
9	*	1.200 D + 0.500 Lp + 1.000 W6
10	*	1.200 D + 0.500 Lp + 1.000 W7
11	*	1.200 D + 0.500 Lp + 1.000 W8
12	*	1.200 D + 0.500 Lp + 1.000 W9
13	*	1.200 D + 0.500 Lp + 1.000 W10
14	*	1.200 D + 0.500 Lp + 1.000 W11
15	*	1.200 D + 0.500 Lp + 1.000 W12
16	*	1.200 D + 0.500 Lp - 1.000 W1
17	*	1.200 D + 0.500 Lp - 1.000 W2
18	*	1.200 D + 0.500 Lp - 1.000 W3

MORE LOAD COMBO NOT
SHOWN FOR CLARITY

RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

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Steel Code: AISC360-16 LRFD

387 * 0.813 D - 1.000 E36

* = Load combination currently selected to use

SAMPLE BRACED FRAME
SECTION STEEL
STANDARD CODE CHECK

Frame #1:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
12	14.81	0.00	0.00	-0.21	-0.00	1	0.02 Axial	50	W14X68
13	15.78	0.00	0.00	-0.21	0.00	1	0.03 Axial	50	W14X68

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
31	3.04	11.74	0.00	-3.43	0.00	134	0.01 H1-1b	50	W24X131

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
22	12.51	0.00	0.00	-0.00	-0.00	205	0.67 Axial	50	HSS3.5X3.5X1/4
23	12.59	0.00	0.00	-0.00	-0.00	134	0.67 Axial	50	HSS3.5X3.5X1/4

Level: Level 3

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
13	97.32	0.00	0.00	0.22	-0.40	133	0.15 Axial	50	W14X68
14	114.53	0.00	0.00	0.22	-0.41	134	0.18 Axial	50	W14X68

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
35	0.00	196.49	0.00	-27.39	0.00	2	0.36 H1-1b	50	W18X71

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
18	45.20	0.00	0.00	-0.00	-0.00	134	0.52 Axial	50	HSS5X5X3/8
19	41.84	0.00	0.00	0.00	0.00	133	0.48 Axial	50	HSS5X5X3/8

Level: Level 2

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
13	138.95	1.87	1.59	-0.19	0.00	133	0.23 H1-1a	50	W14X68
14	167.44	-0.25	-1.63	-0.13	0.41	134	0.27 H1-1a	50	W14X68

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
36	-0.96	-44.07	0.00	-16.78	0.00	2	0.08 H1-1b	50	W18X71

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RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

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Steel Code: AISC360-16 LRFD

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
10	75.44	0.00	0.00	0.00	0.00	133	0.87 Axial	50	HSS5X5X3/8
11	77.16	0.00	0.00	0.00	0.00	134	0.89 Axial	50	HSS5X5X3/8

SAMPLE MOMENT FRAME
SECTION STEEL
STANDARD CODE CHECK

Frame #2:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
18	14.65	21.69	-0.00	-2.33	-0.00	115	0.06 H1-1b	50	W18X71
35	14.14	34.35	0.00	-2.88	0.00	115	0.04 H1-1b	50	W18X119
36	10.32	31.13	0.00	3.36	-0.00	2	0.06 H1-3a(H1-1b)	50	W18X71

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
126	1.76	-27.04	0.00	-4.91	0.00	112	0.06 H1-3a(H1-1b)	50	W14X68
127	3.07	-33.55	-0.00	6.46	0.00	115	0.08 H1-3a(H1-1b)	50	W14X68

Level: Level 3

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
20	77.72	87.01	-0.00	-11.53	0.01	115	0.24 H1-1b	50	W18X71
44	66.70	141.87	0.00	-18.33	0.01	115	0.17 H1-3a(H1-1b)	50	W18X119
45	63.12	-79.53	0.00	10.42	0.00	112	0.20 H1-3b	50	W18X71

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
140	0.00	-115.98	0.00	-21.77	0.00	112	0.21 H1-1b	50	W21X62
141	0.00	-146.75	0.00	31.88	-0.00	115	0.27 H1-1b	50	W21X62

Level: Level 2

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
26	112.76	208.80	0.18	-16.22	-0.01	115	0.25 H1-3a(H1-1b)	50	W18X119
32	115.81	-110.23	0.04	8.56	-0.00	112	0.36 H1-3b	50	W18X71
48	144.70	116.25	0.09	-9.03	-0.01	115	0.44 H1-3b	50	W18X71

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
71	0.00	-210.42	0.00	34.03	0.00	115	0.39 H1-1b	50	W21X62

RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

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Steel Code: AISC360-16 LRFD

Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
149	0.00	-177.77	0.00	-27.20	0.00	112	0.33 H1-1b	50	W21X62

MOMENT FRAME DESIGN

Seismic Provisions Member Code Check Summary

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Frame #2:

Story:Roof Column: 18 Type: Special Moment Resisting Frame

Size:W18X71 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

SAMPLE SEISMIC
PROVISION CHECK
FOR MOMENT FRAME**Story:Roof Column: 35 Type: Special Moment Resisting Frame**

Size:W18X119 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Roof Column: 36 Type: Special Moment Resisting Frame

Size:W18X71 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Roof Beam:126 Type: Special Moment Resisting Frame

Size:W14X68 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story:Roof Beam:127 Type: Special Moment Resisting Frame

Size:W14X68 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story:Level 3 Column: 20 Type: Special Moment Resisting Frame

Size:W18X71 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Level 3 Column: 44 Type: Special Moment Resisting Frame

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Size: W18X119 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 3**Column: 45****Type: Special Moment Resisting Frame**

Size: W18X71 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 3**Beam: 140****Type: Special Moment Resisting Frame**

Size: W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story: Level 3**Beam: 141****Type: Special Moment Resisting Frame**

Size: W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story: Level 2**Column: 26****Type: Special Moment Resisting Frame**

Size: W18X119 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 2**Column: 32****Type: Special Moment Resisting Frame**

Size: W18X71 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 2**Column: 48****Type: Special Moment Resisting Frame**

Size: W18X71 Fy (ksi): 50.00

D1.4a Required Strength --- OK

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Level 2 Beam: 71 Type: Special Moment Resisting Frame

Size:W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story:Level 2 Beam:149 Type: Special Moment Resisting Frame

Size:W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

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MOMENT FRAME JOINT SEISMIC PROVISION CHECK

RAM Structural System

Seismic Provisions Joint Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

11/22/20 21:35:21

Steel Code: AISC341-16 - LRFD

Criteria

Use frame numbers to designate a single line of bracing

 $C_d/I_e = 5.0$

Apply AISC 358 Provisions to Moment Frame

RBS Strain Hardening Factor $C_{pr} = 1.10$ Other Strain Hardening Factor $C_{pr} = 1.40$

AISC 358 Live Load Factor = 0.50

AISC 358 Snow Load Factor = 0.20

BRBF and SCBF - Required Column Strength based on Larger of Strength and Capacity Limited Seismic Forces.

Frame #2:

Story:Roof Joint: 7 Type: Special Moment Resisting Frame

Size:W18X71 Fy (ksi): 50.00

E3.4a Moment Ratio --- OK

E3.4c --- Stability Bracing at Beam-to-Column Connections

E3.6d Connection Required Shear Strength

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- OK With Web Plate / Stiffeners

E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- OK

E3.6f Continuity Plates --- OK With Web Plate / Stiffeners

Story:Roof Joint: 12 Type: Special Moment Resisting Frame

Size:W18X119 Fy (ksi): 50.00

E3.4a Moment Ratio --- OK

E3.4c --- Stability Bracing at Beam-to-Column Connections

E3.6d Connection Required Shear Strength

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- OK With Web Plate / Stiffeners

E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- OK

E3.6f Continuity Plates --- OK With Web Plate / Stiffeners

Story:Roof Joint: 14 Type: Special Moment Resisting Frame

Size:W18X71 Fy (ksi): 50.00

E3.4a Moment Ratio --- OK

E3.4c --- Stability Bracing at Beam-to-Column Connections

E3.6d Connection Required Shear Strength

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- OK With Web Plate / Stiffeners

E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- OK

E3.6f Continuity Plates --- OK With Web Plate / Stiffeners

Story:Level 3 Joint: 24 Type: Special Moment Resisting Frame

Size:W18X71 Fy (ksi): 50.00

E3.4a Moment Ratio --- OK

E3.4c --- Stability Bracing at Beam-to-Column Connections

E3.6d Connection Required Shear Strength

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- OK With Web Plate / Stiffeners

E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- OK

RAM Structural System

Seismic Provisions Joint Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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11/22/20 21:35:21

Steel Code: AISC341-16 - LRFD

E3.6f Continuity Plates --- [OK With Web Plate / Stiffeners](#)**Story:Level 3 Joint: 28 Type: Special Moment Resisting Frame**

Size:W18X119 Fy (ksi): 50.00

E3.4a Moment Ratio --- [OK](#)

E3.4c --- Stability Bracing at Beam-to-Column Connections

E3.6d Connection Required Shear Strength

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- [OK With Web Plate / Stiffeners](#)E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- [OK](#)E3.6f Continuity Plates --- [OK With Web Plate / Stiffeners](#)**Story:Level 3 Joint: 30 Type: Special Moment Resisting Frame**

Size:W18X71 Fy (ksi): 50.00

E3.4a Moment Ratio --- [OK](#)

E3.4c --- Stability Bracing at Beam-to-Column Connections

E3.6d Connection Required Shear Strength

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- [OK With Web Plate / Stiffeners](#)E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- [OK](#)E3.6f Continuity Plates --- [OK With Web Plate / Stiffeners](#)**Story:Level 2 Joint: 45 Type: Special Moment Resisting Frame**

Size:W18X119 Fy (ksi): 50.00

E3.4a Moment Ratio --- [OK](#)

E3.4c --- Stability Bracing at Beam-to-Column Connections

E3.6d Connection Required Shear Strength

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- [OK With Web Plate / Stiffeners](#)E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- [OK](#)E3.6f Continuity Plates --- [OK With Web Plate / Stiffeners](#)**Story:Level 2 Joint: 47 Type: Special Moment Resisting Frame**

Size:W18X71 Fy (ksi): 50.00

E3.4a Moment Ratio --- [OK](#)

E3.4c --- Stability Bracing at Beam-to-Column Connections

E3.6d Connection Required Shear Strength

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- [OK With Web Plate / Stiffeners](#)E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- [OK](#)E3.6f Continuity Plates --- [OK With Web Plate / Stiffeners](#)**Story:Level 2 Joint: 40 Type: Special Moment Resisting Frame**

Size:W18X71 Fy (ksi): 50.00

E3.4a Moment Ratio --- [OK](#)

E3.4c --- Stability Bracing at Beam-to-Column Connections

E3.6d Connection Required Shear Strength

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- [OK With Web Plate / Stiffeners](#)E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- [OK](#)E3.6f Continuity Plates --- [OK With Web Plate / Stiffeners](#)

Story Number: 1

Joint Number: 45

Final Design

No Web Plate Required

No Top Flange Stiffener Required

No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) _____ 50.00

Stiffener Nominal Yield (ksi) _____ 50.00

	<u>Size</u>	<u>Plan Angle</u>	<u>Elev Angle</u>	<u>Yield(ksi)</u>
Col. At Jnt:	W18X119	0.00	---	50.00
Beam SideA :	W21X62	0.00	0.00	50.00
Beam SideB :	W21X62	180.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.

Use actual beam moments to determine panel zone shear at the joint.

Design stiffeners to all be the same size at a joint

Results

Panel Zone

<u>Side</u>	<u>Moment</u> (kip-ft)	<u>Axial</u> (kip)	<u>Shear</u> (kip)	<u>Load Combination</u>
A	-206.90	0.00	33.79	1.287 D + 0.500 Lp + 1.000 E16
B	75.90	0.00	2.26	1.287 D + 0.500 Lp + 1.000 E16
Shear Force In Column Above Joint(kip)				= -19.13
Controlling Shear Force (kip)				= 151.46
Column Web Capacity w/o Web Plate (kip)				= 384.99

Compression

	<u>Flange</u>	<u>Force</u> (kip)	<u>LCo</u>	<u>Cap.</u> (kip)	<u>Force</u> (kip)	<u>LCo</u>	<u>Cap.</u> (kip)	<u>Stiffen</u>
Local Web Yld	Top	40.8	328	259.2	59.9	331	259.2	NO
	Bot	121.8	115	259.2	102.7	112	259.2	NO
Web Crippling	Top	40.8	328	412.9	59.9	331	412.9	NO
	Bot	121.8	115	412.9	102.7	112	412.9	NO
Web Buckling	Bot	77.4	2	454.5	43.4	2	454.5	NO

Tension

	<u>Flange</u>	<u>Force</u> (kip)	<u>LCo</u>	<u>Cap.</u> (kip)	<u>Force</u> (kip)	<u>LCo</u>	<u>Cap.</u> (kip)	<u>Stiffen</u>
Local Web Yld	Top	121.8	115	259.2	102.7	112	259.2	NO
	Bot	40.8	328	259.2	59.9	331	259.2	NO
Flange Bend.	Top	121.8	115	316.0	102.7	112	316.0	NO
	Bot	40.8	328	316.0	59.9	331	316.0	NO

MOMENT FRAME JOINT SAMPLE CALC Seismic Provisions Joint Code Check

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

02/24/21 10:51:59

Steel Code: AISC341-16 - LRFD

Joint Parameters

Story: Level 2 Frame No: 2 Joint No: 45
Fy (ksi): 50.00 Column Size: W18X119
Joint Frame Type: Special Moment Resisting Frame
Apply AISC 358 Provisions to Moment Frame
Strain Hardening Factor Cpr = 1.40

E3.4a Moment Ratio --- OK

Col.	Size	M*pc (kip-ft)	Zc (in ³)	Pu (kip)	Combo#
Top	W18X119	1047.12	262.00	71.62	13
Bot	W18X119	1012.90	262.00	126.63	13
Bm.	Size	M*pb (kip-ft)	Mpr (kip-ft)	Muv (kip-ft)	Sh+dc/2 (in)
71	W21X62	1008.82	924.00	84.82	9.35
149	W21X62	997.85	924.00	73.85	9.35

Sh = Distance face-of-column to location of hinge.

Mpr = Cpr x Ry x Fy x Zhng, where Zhng = Z at hinge.

$\Sigma M^*_{pc} = \Sigma Z_c(F_y - P_u/A_g)$ (kip-ft) = 2060.02 ΣM^*_{pb} (kip-ft) = 2006.67

$\Sigma M^*_{pc} / \Sigma M^*_{pb}$ (Eqn E3-1) = 1.03 > 1.0 OK

E3.4c Stability Bracing at Beam-to-Column Connections --- OK

Joint is restrained in the minor axis at both the top and bottom flange of the moment-frame beams.

Note! RAM Frame assumes that a beam framing into the minor axis of the joint, braces the column at the elevation of both top and bottom flange of the girders connected to the flanges of the column.

E3.6d Connection Required Shear Strength

Bm No.	Col Side	Mpr (kip-ft)	L (ft)	0.8Mp(kip-ft)	V Max (kip)
71	Flange	924.00	20.450	480.00	108.86
149	Flange	924.00	17.450	480.00	117.04

V Max = Max shear from applicable load combinations with Ecl

Mpr = Cpr x Ry x Fy x Zhng, where Zhng = Z at hinge.

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- OK With Web Plate / Stiffeners

Bm.	Size	Bf Red. (in)	Z (in ³)	Z hng (in ³)	Mpr (kip-ft)
71	W21X62	0.000	144.00	144.00	924.00
149	W21X62	0.000	144.00	144.00	924.00
Bm.	Mpr (kip-ft)	Sh (in)	V hng (kip)	Mf (kip-ft)	Vpz (kip)
71	924.00	0.00	108.862	924.00	543.93
149	924.00	0.00	117.039	924.00	543.93

Sh = Distance face-of-column to location of hinge.

Vhng = Shear at hinge or col face from applicable load combinations with Ecl

Mpr = Cpr x Ry x Fy x Zhng, where Zhng = Z at hinge.

Required Panel Zone Shear Strength (kip) = 953.46

Panel Zone Shear Reduced by Vc (kip) = 134.40

Axial Comp. for Panel Zone Calc. (kip) = 126.63 - Combination 1.287 D + 0.500 Lp - 2.000 E16

Panel Zone Strength Without Web Plate (kip) = 427.76 NG

Web Plate Reqd Thickness (in) = 0.922

Web Plate Thickness Used (in) = 1.000

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RAM Structural System

Seismic Provisions Joint Code Check

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Web plate thickness req'd to fillet weld to column flanges (in) = 0.213

Panel Zone Strength With Web Plate (kip) = 997.76 OK

E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- OK

No plug welds specified so both column and web plate must meet Equation E3-7

dz (in) = 19.770 wz (in) = 16.880

Column web thickness reqd (in) = 0.407 Thickness provided = 0.655 OK

Web PL thickness reqd (in) = 0.407 Thickness provided = 1.000 OK

358 - 2.4.4 Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
71	W21X62	50.00	8.24	0.615	1.37	1.06	Yes
149	W21X62	50.00	8.24	0.615	1.37	1.06	Yes

See Additional AISC 358 Requirements for Assigned Connection Type.

BRACED FRAME DESIGN

Seismic Provisions Member Code Check Summary

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

04/23/21 15:46:27

Steel Code: AISC341-16 - LRFD

Criteria

Use frame numbers to designate a single line of bracing

Cd/Ie = 5.0

Apply AISC 358 Provisions to Moment Frame

RBS Strain Hardening Factor Cpr = 1.10

Other Strain Hardening Factor Cpr = 1.40

AISC 358 Live Load Factor = 0.50

AISC 358 Snow Load Factor = 0.20

BRBF and SCBF - Required Column Strength based on Larger of Strength and Capacity Limited Seismic Forces.

SAMPLE SEISMIC
PROVISION CHECK
FOR BRACED FRAME

Frame #1:

Story:Roof **Column: 12** **Type: Special Concentric Brace Frame -Chevron**

Size:W14X68 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story:Roof **Column: 13** **Type: Special Concentric Brace Frame -Chevron**

Size:W14X68 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

ADDITIONAL CHECK WILL
BE SHOWN AFTER
SEISMIC PROVISION
CHECK SUMMARY**Story:Roof** **Beam: 31** **Type: Special Concentric Brace Frame -Chevron**

Size:W24X131 Fy (ksi): 50.00

F2.4b V- and Inverted V- Braced Frames --- Additional Check Required

F2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Story:Roof **Brace: 22** **Type: Special Concentric Brace Frame -Chevron**

Size:HSS3.5X3.5X1/4 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- OK

F2.5 Bracing Members --- OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

F2.5b(1) Slenderness --- OK

Story:Roof **Brace: 23** **Type: Special Concentric Brace Frame -Chevron**

Size:HSS3.5X3.5X1/4 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- OK

F2.5 Bracing Members --- OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

F2.5b(1) Slenderness --- OK

Story:Level 3 **Column: 13** **Type: Special Concentric Brace Frame -Chevron**

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Size: W14X68 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 3**Column: 14****Type: Special Concentric Brace Frame -Chevron**

Size: W14X68 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 3**Beam: 35****Type: Special Concentrically Braced Frame**

Size: W18X71 Fy (ksi): 50.00

F2.3 Non-V and Non-Inverted V-Braced Frame - Unbalanced Forces Design Required.

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 3**Brace: 18****Type: Special Concentric Brace Frame -Chevron**

Size: HSS5X5X3/8 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- OK

F2.5 Bracing Members --- OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

F2.5b(1) Slenderness --- OK

Story: Level 3**Brace: 19****Type: Special Concentric Brace Frame -Chevron**

Size: HSS5X5X3/8 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- OK

F2.5 Bracing Members --- OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

F2.5b(1) Slenderness --- OK

Story: Level 2**Column: 13****Type: Special Concentric Brace Frame -Chevron**

Size: W14X68 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 2**Column: 14****Type: Special Concentric Brace Frame -Chevron**

Size: W14X68 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Story:Level 2 Beam: 36 Type: Special Concentric Brace Frame -Chevron

Size:W18X71 Fy (ksi): 50.00

F2.4b V- and Inverted V- Braced Frames --- **Additional Check Required**F2.5a Basic Requirements (D1.1 Moderately Ductile) --- **OK****Story:Level 2 Brace: 10 Type: Special Concentric Brace Frame -Chevron**

Size:HSS5X5X3/8 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- **OK**F2.5 Bracing Members --- **OK**F2.5a Basic Requirements (D1.1 Highly Ductile) --- **OK**F2.5b(1) Slenderness --- **OK****Story:Level 2 Brace: 11 Type: Special Concentric Brace Frame -Chevron**

Size:HSS5X5X3/8 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- **OK**F2.5 Bracing Members --- **OK**F2.5a Basic Requirements (D1.1 Highly Ductile) --- **OK**F2.5b(1) Slenderness --- **OK**This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v3B>

SCBF BEAM ADDITIONAL CHECK

Per AISC Seismic Provision Section F2.3(i) and F2.3(ii)

Roof Beam

Try W24X131 $L_p := 29.33 \text{ ft}$
 $P_n := 1624.91$ Kips
 $M_{nx} := 1387.5$ Kips - ft
 $M_{ny} := 305.6$ Kips - ft

F2.3(a)

With Unbalanced Load:

Vertical Resultant Horizontal Resultant
 $E_{mv} := 127.3$ Kips $E_{mh} := 165.4$ Kips

Axial Load

$$P_u := 3.03 + E_{mh} = 168.43 \text{ Kips}$$

Moment:

$$M_{ux} := 7.26 + E_{mv} \cdot \frac{L}{4} = 940.687 \text{ Kips - ft}$$

$$M_{uy} := 0 \text{ Kips - ft}$$

Check P_u/P_n

$$\frac{P_u}{P_n} = 0.104$$

$P_r/P_c > 0.2$ Use Equation

$$\frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad H1 - 1a$$

$P_r/P_c < 0.2$ Use Equation

$$\frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad H1 - 1b$$

Interaction Check

$$\text{Int_Check} := \begin{cases} \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} > 0.2 \\ \frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} \leq 0.2 \end{cases}$$

$$\text{Int_Check} = 0.73$$

F2.3(b)

With Unbalanced Load:

Vertical Resultant

$$E_{mv} := 140.6 \quad \text{Kips}$$

Horizontal Resultant

$$E_{mh} := 152. \quad \text{Kips}$$

Axial Load

$$P_u := 3.03 + E_{mh} = 155.03 \quad \text{Kips}$$

Moment:

$$M_{ux} := 7.26 + E_{mv} \cdot \frac{L}{4} = 1.038 \times 10^3 \quad \text{Kips} - \text{ft}$$

$$M_{uy} := 0 \quad \text{Kips} - \text{ft}$$

Check P_u/P_n

$$\frac{P_u}{P_n} = 0.095$$

$P_r/P_c > 0.2$ Use Equation

$$\frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad \text{H1 - 1a}$$

Interaction Check

$$\text{Int_Check} := \begin{cases} \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} > 0.2 \\ \frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{otherwise} \end{cases}$$

$P_r/P_c < 0.2$ Use Equation

$$\frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad \text{H1 - 1b}$$

$$\text{Int_Check} = 0.796$$

W24X131 OK

SCBF Beam Additional Check

Per AISC Seismic Provision Section F2.3(i) and F2.3(ii)

Third Floor Beam

Try W18x71 $L := 29.3$ ft

$P_n := 936$ Kips

$M_{nx} := 547.5$ Kips - ft

$M_{ny} := 92.63$ Kips - ft

F2.3(b) By inspection, we know this will govern

With Unbalanced Load:

From Member Force in Ram,

$F_{upper} := 206.98$ Kips For Expected Tension of Brace 3.5x3.5x5/16

$F_{lower} := 32.8$ Kips For Expected Post Buckling Comp of Brace 5 x 5 x 3/8

$F_{Diaphragm} := 35$ Kips From Ram Structure

$\theta_1 := 44.705$

$\theta_2 := 43.288$

$E_{mh} := F_{upper} \cdot \cos(\theta_1 \text{ deg}) - F_{lower} \cdot \cos(\theta_2 \text{ deg}) + F_{Diaphragm} = 158.233$ Kips

Vertical Resultant

$$E_{mv} := 0$$

Kips

Horizontal Resultant

$$E_{mh} = 158.233 \text{ Kips}$$

Axial Load

$$P_u := 3.03 + E_{mh} = 161.263 \text{ Kips}$$

Moment:

$$M_{ux} := 35.5 + E_{mv} \cdot \frac{L}{4} = 35.5 \text{ Kips} - \text{ft}$$

$$M_{uy} := 0 \text{ Kips} - \text{ft}$$

Check P_u/P_n

$$\frac{P_u}{P_n} = 0.172$$

$P_r/P_c > 0.2$ Use Equation

$$\frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad H1 - 1a$$

Interaction Check

$$Int_Check := \begin{cases} \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} > 0.2 \\ \frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{otherwise} \end{cases}$$

$P_r/P_c < 0.2$ Use Equation

$$\frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad H1 - 1b$$

$$Int_Check = 0.151$$

W18x71 OK

Second Floor Beam

Since the Brace at top and bot are the same(Size and Geometry), Unblanced Load can be ignored, use W18x71

BRACED FRAME CONNECTION(GUSSET) CHECK SAMPLE CALC:

Bay Size

$L = 14.665$ ft
 $H_{lower} = 13.75$ ft
 $\theta_{lower} = 46.8443614$ deg.
 $H_{upper,R} = 13.75$ ft
 $\theta_{upper,R} = 43.1556386$ deg.
 $H_{upper,L} = 14.583$ ft
 $\theta_{upper,L} = 44.8393654$ deg.

Members

Beam = W18X65
 Slab Thk. = 5.5 in

Mat'l:

Gusset PL = A36
 Brace = A500-B SQ & RECT
 Reinfr. PL = A572-50
 Beam = A992
 Column = A500-B SQ & RECT

Doubler PL Details:

$t_{PL} = 0.125$ in
 $h = 8$ in

Upper Brace

HSS5X5X3/8

Gusset Details:

$\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 1$ in.
 $t_g = 0.75$ in.
 $L_w = 17$ in.
 Pull-off (R) = 32 in.
 Pull-off (L) = 32 in.
 Clip = 1 in.
 Over Slot = 2 in.
 $D_{brace} = 5$
 $D_{beam} = 5$

Brace Reinforcing Details:

$t_{PL} = 0.4375$ in.
 $b = 4$ in.
 $L_w = 10$ in.
 $D_{brace} = 4$

$L = 0$ in
 $D_{brace} = 4$

Lower Brace

HSS5X5X3/8

Gusset Details:

$\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 1$ in.
 $t_g = 0.75$ in.
 $L_w = 17$ in.
 Pull-off = 32 in.
 Clip = 1 in.
 Over Slot = 2 in.
 $D_{brace} = 5$
 $D_{beam} = 5$

Brace Reinforcing Details:

$t_{PL} = 0.4375$ in.
 $b = 4$ in.
 $L_w = 10$ in.
 $D_{brace} = 4$

$L_{min} = 4.407$ in

Analysis

Welds $F_{EXX} = 70$ ksi
 Weld Ductility Factor = 1.25
 Gusset K Factor = 0.65
 Drag DF = 0.5
 Doubler PL OK? Yes

Status

DESIGN OK!

Upper Gusset

Gusset Edge Forces

Left Brace:

$R_v F_v A_g = 397.992$ k
 $1.14 F_{cre} A_g = 180.4141$ k
 $0.3 F_{cre} A_g = 54.12422$ k
 Right Brace:
 $R_v F_v A_g = 397.992$ k
 $1.14 F_{cre} A_g = 193.9198$ k
 $0.3 F_{cre} A_g = 58.17595$ k
 $N_u = 145.0057$ k
 $V_u = 418.2637$ k
 $M_u = 3848.026$ in-k
 $N_{u,eq} = 399.4607$ k
 $N_u = 234.0554$ k
 $V_u = 328.7133$ k
 $M_u = 3024.162$ in-k
 $N_{u,eq} = 434.0315$ k

Gusset Geometry

$L_{Left} = 28.13$ in
 $L_{Right} = 28.58$ in
 $LA = 60.49$ in
 $LB = 18.21$ in
 $Lg, max = 16.62$ in

Lower Gusset

Gusset Edge Forces

$R_v F_v A_g = 397.992$ k
 $1.14 F_{cre} A_g = 193.9198$ k
 $0.3 F_{cre} A_g = 58.17595$ k
 $N_u = 139.5818$ k
 $V_u = 431.7987$ k
 $M_u = 3972.548$ in-k
 $N_{u,eq} = 399.6915$ k
 $N_u = 232.4282$ k
 $V_u = 332.7738$ k
 $M_u = 3061.519$ in-k
 $N_{u,eq} = 432.8867$ k

Gusset Geometry

$L = 28.58$ in
 $LA = 61.09$ in
 $LB = 18.08$ in
 $Lg = 16.62$ in

Limit States

	Spec Eqn	R_n	ϕR_n	$R_u / \phi R_n$	
<u>Upper Brace</u>					
Brace Weld	J2-4	398.0	473.3	0.84	OK
Net Sect. Rupt.	J4-2	398.0	412.1	0.97	OK
Shear Yield	J4-3	398.0	917.0	0.43	OK
Shear Rupture	J4-4	398.0	743.3	0.54	OK
Reinf. Weld	J2-4	96.3	111.4	0.86	OK
<u>Upper Brace-Gusset</u>					
Tension Yield	J4-1	398.0	598.5	0.66	OK
Buckling	E3	213.3	524.9	0.41	OK
Block Shear	J4-5	398.0	576.2	0.69	OK
<u>Upper Gusset-Beam</u>					
Shear Yield	J4-3	418.3	979.9	0.43	OK
Tension Yield	J4-1	399.5	1469.9	0.27	OK
		434.0	1469.9	0.30	OK
Flexural Yield	F11-1	3848.0	22229.0	0.17	OK
Interaction		0.22	1.0	0.22	OK
		0.17	1.0	0.17	OK
Beam Weld	J2-4	723.0	1083.8	0.67	OK
		680.6	1141.8	0.60	OK
WLY	J10-2	399.5	1490.4	0.27	OK
		434.0	1490.4	0.29	OK
WC	J10-4	399.5	1054.6	0.38	OK
		434.0	1054.6	0.41	OK
<u>Lower Brace</u>					
Brace Weld	J2-4	398.0	473.3	0.84	OK
Net Sect. Rupt.	J4-2	398.0	412.1	0.97	OK
Shear Yield	J4-3	398.0	917.0	0.43	OK
Shear Rupture	J4-4	398.0	743.3	0.54	OK
Reinf. Weld	J2-4	96.3	111.4	0.86	OK
<u>Lower Brace-Gusset</u>					
Tension Yield	J4-1	398.0	598.5	0.66	OK
Buckling	E3	213.3	524.9	0.41	OK
Block Shear	J4-5	398.0	576.2	0.69	OK
<u>Lower Gusset-Beam</u>					
Shear Yield	J4-3	431.8	989.7	0.44	OK
Tension Yield	J4-1	399.7	1484.5	0.27	OK
		432.9	1484.5	0.29	OK
Flexural Yield	F11-1	3972.5	22672.1	0.18	OK
Interaction		0.22	1.0	0.22	OK
		0.17	1.0	0.17	OK
Beam Weld	J2-4	735.5	1088.5	0.68	OK
		682.5	1150.6	0.59	OK
WLY	J10-2	399.7	1503.9	0.27	OK
		432.9	1503.9	0.29	OK
WC	J10-4	399.7	1063.2	0.38	OK
		432.9	1063.2	0.41	OK
<u>Beam</u>					
Shear Yield	G2-1	260.0	278.4	0.93	OK
		201.0	278.4	0.72	OK
Doubler Weld	J2-4	1.43	8.35	0.17	OK

END

1987 NBC “Risk Category III” “Seismic Design Category C”

This section presents representative structural calculations for Building 3 (3-story education facility in St. Louis, Missouri) Design Case C. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v3A.

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DESIGN CRITERIA

DESIGN CRITERIA

VERTICAL LOADS

AREA	DESIGN DEAD LOAD	LIVE LOAD	CONCENTRATED LOADS
CLASSROOM AND OFFICE	40 PSF	50 PSF	1000#
CORRIDORS ABOVE 1ST FLOOR	40 PSF	80 PSF	1000#
ROOF	20 PSF	20 PSF	-

SNOW:

$P_g = 20$ PSF = GROUND SNOW LOAD

$P_f = C_e I P_g$ = FLAT ROOF SNOW LOAD = 15.4 PSF, USE 20 FOR CALCULATION

$I = 1.1$, $C_e = 1.0$

LATERAL FORCES

LATERAL FORCES ARE TRANSMITTED BY DIAPHRAGM ACTION OF ROOF AND FLOORS TO BRACED FRAME/MOMENT FRAME. LOADS ARE THEN TRANSFERRED TO FOUNDATION BY BRACED FRAME/MOMENT FRAME ACTION WHERE ULTIMATE DISPLACEMENT IS RESISTED BY PASSIVE PRESSURE OF EARTH AND/OR SLIDING FRICTION. OVERTURNING IS RESISTED BY DEAD LOAD OF THE STRUCTURE.

WIND:

THE BUILDING MEETS THE CRITERIA PER NBC 1987 SECTION 1112.0.

- EXPOSURE CATEGORY = B
- BASIC WIND SPEED, $V = 70$ MPH
- EQUIVALENT RISK CATEGORY PER TABLE 1.5-1 = III
- PRESSURE COEFFICIENT (ENCLOSED) = 0.8, -0.5
- WIND IMPORTANCE FACTOR $I_w = 1.07$
- BRACE FRAME BASE SHEAR $V = 89.9$ KIPS; MOMENT FRAME BASE SHEAR $V = 63.5$ KIPS

SEISMIC: (NBC 1987) $V = Z I_k C_{SW}$

$Z = 3/8$

SEISMIC IMPORTANCE FACTOR, $I = 1.25$

EQUIVALENT RISK CATEGORY PER TABLE 1113.1 = III

$k = 1.0$ AT BRACED FRAME, $k = 1.0$ AT MOMENT FRAME

$W =$ EFFECTIVE SEISMIC WEIGHT OF BUILDING = 2340 KIPS

ANALYSIS PROCEDURE USED = EQUIVALENT LATERAL FORCE PROCEDURE

DESIGN BRACE FRAME BASE SHEAR $V = 154$ KIPS

DESIGN MOMENT FRAME BASE SHEAR $V = 144$ KIPS

FOUNDATION DESIGN CRITERIA

SOIL BEARING PRESSURE: 4000 PSF (ASSUMED)

ACTIVE PRESSURE - RESTRAINED: 50 PCF +14H SEISMIC SURCHARGE (ASSUMED)

ACTIVE PRESSURE - UNRESTRAINED: 35 PCF +6H SEISMIC SURCHARGE (ASSUMED)

PASSIVE RESISTANCE: 200 PCF (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

COEFFICIENT OF FRICTION: .35 (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

*1/3 INCREASE ALLOWED FOR SEISMIC OR WIND LOADING

CONCRETE

CAST-IN-PLACE CONCRETE

ITEM	DESIGN f'_c (PSI) (AT 28 DAYS U.N.O.)
FOUNDATIONS	3000
SLABS ON GRADE AND SLABS ON METAL DECK	4000

REINFORCING STEEL

ASTM A615, GRADE 60 TYPICAL UNLESS NOTED OTHERWISE.

REINFORCING SPLICE AND DEVELOPMENT LENGTH SCHEDULE, $F_y=40$ KSI (UNLESS NOTED OTHERWISE)

BAR SIZE	MINIMUM LAP SPLICE LENGTHS ("Ls")	MINIMUM DEVELOPMENT LENGTHS ("Ld")
#3	1'-6"	1'-3"
#4	2'-0"	1'-7"
#5	2'-7"	2'-0"
#6	3'-1"	2'-4"
#7	4'-6"	3'-6"
#8	5'-2"	3'-11"
#9	5'-10"	4'-6"
#10	6'-6"	5'-0"
#11	7'-3"	5'-7"

STRUCTURAL STEEL

MATERIAL PROPERTIES

WIDE FLANGE SECTIONS: ASTM A36 ($F_y = 36$ KSI)

OTHER SHAPES AND PLATES: ASTM A36 ($F_y = 36$ KSI)

STRUCTURAL TUBING: RECTANGULAR & SQUARE - A500 GRADE BB ($F_y = 46$ KSI)

MACHINE BOLTS (M.B.): ASTM A307

HIGH-STRENGTH BOLTS: A325-ASTM F1852, A490-ASTM F2280

ANCHOR BOLTS (A.B.): ASTM A490, GRADE 36, UNLESS OTHERWISE NOTED

GRAVITY DESIGN

ROOF BEAM DESIGN

Beam # = RB4

Shape W 16x26

Properties	Geometry	
	A	7.67 in. ²
	d	15.65 in.
	b _f	5.5 in.
	t _f	0.345 in.
	t _w	0.25 in.
	W(weight)	26 #/ft.
	r _T	1.38
	d/A _f	8.25

Compact Section Criteria	
b _f /2t _f	7.97
F _y	42.9
d/t _w	62.6
F _y	43.3 ksi
F _y	16.9 ksi

Elastic Properties					
X-X			Y-Y		
I _x	300	in. ⁴	I _y	9.59	in. ⁴
S _x	38.3	in. ³	S _y	3.49	in. ³
r _x	6.25	in.	r _y	1.12	in.
Plastic Modulus					
Z _x	44	in. ³	Z _y	5.48	in. ³

Torsional Constant	
J	0.261 in. ⁴

Warping Constant	
C _w	562 in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Note:

Input/ verify data in yellow box:

Non-Composite Beam Design AISC Seventh Edition

Building info:

Steel Properties		Loading	
Beam Length =	29.33 ft.	Dead Load (D)=	20 psf
Beam Spacing (w) =	10 ft.	Live Load(L)=	0 psf
Unbraced Length =	29.33 ft.	Roof Live Load(Lr)=	20 psf
Fy =	36 ksi	Snow Load(S)=	20 psf

Deflection Criteria		
Total Load	L/360=	0.98 in.

Max Moment:	
MD(Max Dead)=	258.07 kips-in
MD(Dead Overwrite)=	0.00 kips-in
MD(Max Use)=	258.07 kips-in
ML(Max Live)=	0.00 kips-in
ML(Live Overwrite)=	0.00 kips-in
ML(Max Use)=	0.00 kips-in
M(Lr or S)(Max Live)=	258.07 kips-in
M(Lr or S)(Live Overwrite)=	0.00 kips-in
M(Lr or S)(Max Use)=	258.07 kips-in
Mu=	516.15 kips-in

Controlling Case = D+L+(Lr or S)

Max Shear	
VD(Max Dead)=	2.93 kips
VD(Dead Overwrite)=	0.00 kips
VMD(Max Use)=	2.93 kips
VL(Max Live)=	0.00 kips
VL(Live Overwrite)=	0.00 kips
VL(Max Use)=	0.00 kips
V(Lr or S)(Max Live)=	2.93 kips
V(Lr or S)(Live Overwrite)=	0.00 kips
V(Lr or S)(Max Use)=	2.93 kips
Vu=	5.87 kips

Controlling Case = D+L+(Lr or S)

Solution Summary:		
Try	W 16x26	<---- Change W shapes
Member	Checked Item	DCR
	Beam Bending	56%
	Beam Shear	10%
	Beam deflection	79%
Overall Check		79%

Typical Bending

Camber =	0 in.
----------	-------

Non-Composite Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced lenth (Lb)= 29.33 ft

Checks for Typical Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f/2t_f \leq 55.2/\sqrt{F_y}$	7.97	8.7	Good
$d/t_w \leq 412/\sqrt{F_y}$	62.6	68.7	Good
$l_b/\leq 76b_f/\sqrt{F_y}$	351.96	69.7	No Good, Use Alternate EQN
$l_b/\leq \frac{20,000}{(d/A_f) F_y}$	351.96	67.4	No Good, Use Alternate EQN
Use Alternate Equation			

Checks for Alternate Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2/\sqrt{F_y} < b_f/2t_f$	7.97	8.7	Good
$b_f/2t_f \leq 95.0/\sqrt{F_y}$	7.97	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_b = 0.66F_y$

Fb= N/A ksi

Alternate Bending Stress Equation: $F_b = F_y [0.733 - 0.0014(\frac{b_f}{t_f})\sqrt{F_y}]$

Fb= 24 ksi

Shear Stress Equation: $F_v = 0.40F_y$

Fv= 14.4 ksi

Calculate Stresses in Beam

Bending Stress:	fb= Mu/Sx=	13.5	ksi	Beam Okay
Shear Stress:	fv= Vu/(tw*d)=	1.5	ksi	Beam Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0.77	in.	Beam Okay

FLOOR BEAM DESIGN

Beam # = **FB4**

Shape **W 16x36**

Properties	Geometry	
	A	10.6 in. ²
	d	15.86 in.
	b _f	6.985 in.
	t _f	0.43 in.
	t _w	0.295 in.
	W(weight)	36 #/ft.
	r _T	1.79 in.
	d/A _f	5.28

Compact Section Criteria	
b _f /2t _f	8.1
F _y	64
d/t _w	53.8
F _y	- ksi
F _y	22.9 ksi

Elastic Properties					
X-X			Y-Y		
I _x	448	in. ⁴	I _y	24.5	in. ⁴
S _x	56.5	in. ³	S _y	7	in. ³
r _x	6.51	in.	r _y	1.52	in.
Plastic Modulus					
Z _x	64	in. ³	Z _y	10.8	in. ³

Torsional Constant	
J	0.54 in. ⁴

Warping Constant	
C _w	- in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Note: Input/ verify data in yellow box:

Composite Beam AISC 8th Edition

Note: The following calculation is basic on the "Steel Tips by Steel Committee of California" dated Jan 1987 and of Composite Design for building construction in AISC 8ed Example 2

Assumption:

Item	Ribs Perpendicular	Ribs Parallel
1. Concrete Area below Top of deck	Neglect	Include
2. Stud Reduction Factor	$\frac{0.85}{(Nr)^{1/2}} \left(\frac{wr}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right)$	$0.6 \left(\frac{wr}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right)$
3. Maximum Stud Spacing	32 in.	Not Specified
4. Deck Welding	16 in.	Not Specified
5 Minimum Width of Rib	2 in.	Depends on Nr

Building info:

Steel			Concrete			Deck		
Beam Length	29.3	ft.	Weight	145	pcf	Depth hr	3	in.
Beam Spacing	10	ft.	Thickness t	5.5	inches	Conc. thickness tc	2.5	in.
Fy	36	ksi	f'c	3000	psi	Stud Dia.	3/4	in.
			E'c	3122	ksi	Stud Height	4.5	in.
			β_1	0.85		Deck Orientation	Perpendicular	
			Light Weight concrete Reduction	1		Deck Width, wr	4.5	in.

Uniform Load		Deflection Criteria	
Dead Load	90 psf	Dead Load	L/240 or 1.5 in.
Live Load	50 psf	Live Load	L/360
Roof Live Load	0 psf		
Snow Load	0 psf		

Uniform Load W/ Combo	
w(D+L+Lr or S)	140 psf

Max Moment:		kips-in	Kips-ft
M(max D)		1159.0	96.6
M(D overwrite)	0.0		0.0
M(Dead use)		1159.0	96.6
M(max D+L)		1802.8	150.2
M(D+L overwrite)	0.0		0.0
M(max use)		1802.8	150.2
Max Shear		kips	
V(max D)		13.185	
V(max D+L)		20.51	
V(D+L overwrite)	0		
V(max use)		20.51	

Solution Summary:

W 16x36 <---- Change W shapes

Partial Composite: **75%**

ASD Result:

DCR	
Concrete Stress	0.168
Steel Stress	0.855
Dead Load Δ	0.790 1.1567 in.
Live Load Δ	0.209 0.2044 in.
Shoring?	Need Shoreing <----Check this
# of studs	20 at each side of max moment
Studs per rib check	OK
Camber=	0.750 in.
Overall	GOOD

Result illustration:

Beam	Stud	Camber
W 16x36	40	0.750
o Δ		
20.51 Reaction		20.51 Reaction

$n=E/E_c$	9
-----------	---

1. Effective Width :

$L/4$	87.9	in.
s	120	in.
$16t+bf$	95.0	in.

Govern b_{eff} : 87.9 in.**2. Required Section Moduli**

$M(D+L+S), S_{tr}$	75.1	in.^3
$M(D \text{ only}), S_s$	48.3	in.^3

3. Composite design section properties**a. Moment of Inertia**

AcI	219.75	
Ac/I	357.555	
$Ac=b(tc)$	219.75	in.^2
$A'c=Ac/n$	24.42	in.^2
$Y_s=d/2$	7.93	in.
$Y_c=d+hr+tc/2$	20.11	in.

Section:	A	Y	AY	ds	Ad^2	I_o
W 16x36	10.6	7.93	84.1	8.49	764.6	448.0
Concrete	24.41667	20.11	491	3.69	331.9	12.7
Sum	35.01667		575.1		1096.5	460.7
					$I_{tr}=$	1557.23 in.^4

 $Y_b=$ 16.42 in.**b. Section Moduli**

			Partial Composite	
Str=	94.82	in.^3	Seff=	89.7 in.^3
St=	315.42	in.^3	Ieff=	1408.6 in.^4

4. Check Stress

a. Concrete				
$f_c=M/n*St$	0.2268	ksi <	1.35	OK
b. Steel				
Total Load, Str=	89.7	ksi >	75.117875	OK
Dead Load, Ss=	56.5	ksi >	48.29	OK
Web shear, Fv=	4.384	ksi <	20	OK

5. Check Deflection

$\Delta = \frac{5wL^4}{384EI} = \frac{ML^2}{1920I}$				
$\Delta D=$	1.156711	in. <	1.5	OK
$\Delta L=$	0.20438	in. <	1.0	OK

6. Check if shore is required

Str max=	87.26	in.^3 >	94.82	Need Shoreing
----------	-------	---------	-------	---------------

7. Shear Connection, Stud

a. Max Stud dia	1.075	in. >	0.75	OK
------------------------	-------	-------	------	----

b. Horizontal shear

$V_h=0.85*f_c*Ac/2$	280.1813	kips	Concrete	
$V_h=As*F_y/2$	190.8	kips	Steel	<----- Govern

Reduction Factor

$$\frac{0.85}{(Nr)^{1/2}} \left(\frac{w_r}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right) \quad \text{or} \quad 0.6 \left(\frac{w_r}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right)$$

Rf1= 0.64

Rf2= 1.000

Rf(use)= 0.638

q per stud= 11.5000

q*Rf= 7.331

<-Lightweight Concrete reduction included

Stud Shear	F'c(From AISC V8 Table 1.11.4) (ksi)		
Connector	3000	3500	4000
0.5	5.1	5.5	5.9
0.625	8	8.6	9.2
0.75	11.5	12.5	13.3
0.875	15.6	16.8	18

Stud Per Rib, Nr	1	
Rib Spacing	6	in.

Number of Studs= 20 At each side of where max moment is

Check Stud Per Rib	20	ft	<	29.3	OK
--------------------	----	----	---	------	----

COLUMN DESIGN

COLUMN B5

Column ID	Roof Load (Kips)		Third Floor		Second Floor		Force at the base
1	0	0	0	0	Fba''+Fb8	12.0	12.0
2	Rba+RB1	11.46	Fba+FB1	50.12	FB8+Fba'	42.4	104.0
3	RBd+RB1	9.17	FBd+FB1	39.64	FB1	20.1	68.9
4	RBb+RB6	18.51	FB4+FBg	53.42	53.42	53.4	125.3
5	Rbi+RB6	14.5	FB4+Fba(Canti)	52.11	52.11	52.1	118.7
6	0		0	0.00	Fba''+Fba''	14.7	14.7
7	Rba+Rba+RB1	17.19	Fba+Fba+FB1	80.15	FB8+Fba'+Fba'+FB1	100.3	197.7
8	RBd+Rbe+RB1+RB2	18.05	Fbd+Fbe_Fb1+FB2	87.36	87.36	87.4	192.8
9	Rbe+RB4	13.44	Fbe+FB4+FBg	93.32	93.32	93.3	200.1
10	RB4+Rbi+Rba	20.7	Fba'+Fba'+FB4	80.57	80.57	80.6	181.8
11	0	0	0	0.00	Fba''+Fbb'+FB8	39.3	39.3
12	Rba+RBb+RB1	14.9	Fba+FBb+FB1	72.45	FB8+Fba'+FBb'+FB1	89.9	177.2
13	RB1+Rbe+RBg+RB2	24.55	FB1+Fbe+FBg+FB2	100.72	100.72	100.7	226.0
14	RB1+Rbe+RBg+RB2	22.55	FB2+Fbe+FBg+FB4	101.14	101.14	101.1	224.8
15	Rba+RBb+RB4	15.04	FB4+Fba+FBb	72.87	72.87	72.9	160.8
16	RB1+RBg+RBg+RB2	26.39	FB1+FBg+FBg+FB2	93.73	93.73	93.7	213.9
17	0	0	0	0.00	FBb''+FBb''+FB8	15.4	15.4
18	RBb+RBb+RB1	12.61	FBb+FBb+FB1	64.75	FB8+Fba'+FBb'+FB1	89.9	167.2
19	RB2+RBg+RBg+RB4	26.53	FBg+FBg+FB2+FB4	94.15	94.15	94.2	214.8
20	RB4+RBb+RBb	12.75	FB4+FBb+FBb	65.17	65.17	65.2	143.1
21	RB1+RBg+Rbe+RB2	24.7	FB1+FBg+Fbe	92.9	92.9	92.9	210.5
22	0	0	0	0	FBb''+FBb''+FB8	15.4	15.4
23	RBb+RBc+RB1	12.61	FBb+FBb+FB1	64.75	FB8+FBb'+FBb'	59.3	136.7
24	RB2+RBg+RBg+RB4	26.53	FB2+FBg+FBg+FB4	94.15	94.15	94.2	214.8
25	RBb+RBc+RB4	12.75	FB4+FBb+FBc	65.17	65.17	65.2	143.1
26	RBc+RB1	9.17	Fbi+FB5	49.32	FBb''+FB8	10.0	68.5
27	RBc+RB(7-ED)	10.22	FBb+FBk	41.63	FB8+FBb'+FBb'+FB1	79.4	131.3
28	RB(7-ED)+Rbe+Rbf+RB2	21.02	FBK+Fbe+FBf+FB2	100.35	86.474	87.5	208.8
29	RBg+RBh+RB4	21.15	FB2+FBg+FBf+FBk	90.84	90.84	90.8	202.8
30	RB4+RBc+RBg	15.18	FBk+FBc+FBf	72.44	72.44	72.4	160.1
31	0	0	0	0	FBb'+FB1	47.5	47.5
32	0	0	0	0	FBb'+FBb'	54.7	54.7
33	RBg+RBg	11.74	FB3+FBf	41.15	31.464	31.5	84.4
34	0	0	FB6	4.29	0	0.0	4.3
35	RB(8-EC)+Rbf	8.91	FB6+FBj+FB3	49.14	0	0.0	58.1
36	RB(8-EC)+RBh+RB(8-C-B.5)	11.51	FBx+FBf	40.81	33.56	33.6	85.9
37	RB(8-C-B.5)+RB(8-B.5-B)	2.1	FB3+ FBf	41.15	31.464	31.5	74.7
38	RB(8-B.5-B)+RBg	7.33	2Kips	2.5	2.5	2.5	12.3

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Member # = B5

Shape W 10x54

Properties	Geometry	
	A	15.8 in. ²
	d	10.09 in.
	b _f	10.03 in.
	t _f	0.615 in.
	t _w	0.37 in.
	W(weight)	54 #/ft.
	r _T	2.75
	d/A _f	1.64

Compact Section Criteria	
b _f /2t _f	8.2
F _y	63.5 ksi
d/t _w	27.3
F _y	— ksi
F _y	— ksi

Elastic Properties				
X-X			Y-Y	
I _x	303	in. ⁴	I _y	103 in. ⁴
S _x	60	in. ³	S _y	20.6 in. ³
r _x	4.37	in	r _y	2.56 in
Plastic Modulus				
Z _x	66.6	in. ³	Z _y	31.3 in. ³

Torsional Constant	
J	1.82 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	13.75 ft.
Beam Spacing (w) =	0 ft.
Tributary Area to Col=	0 ft.^2
K=	1
Unbraced Length =	13.75 ft.

Loading	
Dead Load (D)=	90 psf
Live Load(L)=	50 psf
Roof Live Load(Lr)=	0 psf
Snow Load(S)=	25 psf

Eccentricity on Column	
ex =	2 in.
ey =	0 in.

Deflection Criteria		
Total Load	L/360=	0.46 in.

Max Moment About Major Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Shear	
VD(Max Dead)=	0 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	0 kips
VL(Max Live)=	0 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	0 kips
V(Lr or S)(Max Live)=	0 kips
V(Lr or S)(Live Overwrite)=	0 kips
V(Lr or S)(Max Use)=	0 kips
Vu=	0 kips

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	0 kips
Pu=	200 kips

Solution Summary:	
Try	W 10x54 <---- Change W shapes

Member	Major Axis Beam Bending	0%
	Beam Shear	0%
	Minor Axis Beam Bending	0%
	Axial	75%
	Deflection	0%
	Combined Bending and Axial	75%
Overall Check		75%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 13.75 ft
K= 1
KL= 13.75 ft
KL/r= 64.5

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 17 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 12.7$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (L_b)= 13.75 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	8.2	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	27.3	12.2	No Good, Use Alternate EQN
$l_b \leq 76 b_f \sqrt{F_y}$	165	127	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d / A_f) F_y}$	165	339.6	Good
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	8.2	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	8.2	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$F_{bx} = \text{N/A}$ ksi

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$F_{bx} = 23.9$ ksi

Shear Stress Equation: $F_v = 0.40 F_y$

$F_v = 14.4$ ksi

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$F_{by} = 27$ ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	0	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	0	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.75	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.59	Less than 1.0, Okay

Column Name: **B5**

All Equations can be found on Page 3-99 of AISC 8th edition

Column= **W 10x54**

Column Properties	
<i>b</i>	10.03 in
<i>d</i>	10.09 in

Material Properties	
Footing <i>f'_c</i> =	3 ksi
<i>F_y</i> (Plate)=	36 ksi

Max Axial Load = **200** kips

Required Base Plate Area =	$A_{pl} = \frac{P_a}{0.375 f'_c} =$	177.8	in²
----------------------------	-------------------------------------	--------------	-----------------------

Base Plate Design	
<i>B</i> =	12 in.
<i>N</i> =	12 in.

Actual Bearing Pressure =	$f_p = \frac{P_a}{B * N} =$	1.4	ksi
---------------------------	-----------------------------	------------	------------

Plate Thickness		
$m = \frac{N - 0.95d}{2} =$	1.20725	in
$n = \frac{B - 0.8b}{2} =$	1.988	in
$t = m \sqrt{\frac{f_p}{0.25 F_y}} \leq n \sqrt{\frac{f_p}{0.25 F_y}} =$	0.78	in.

- P* = Total column load, kips
- A* = *B* × *N* = Area of plate, square inches
- F_b* = Allowable bending stress in base plate, ksi
- F_p* = Allowable bearing pressure on support, ksi
- f_p* = Actual bearing pressure
- f'_c* = Compressive strength of concrete, ksi or psi
- t* = Thickness of plate, inches

LATERAL DESIGN

EARTHQUAKE LOAD

St. Louis. located at the **Zone 2** of the Figure 1113.1 in BOCA 1987

Per BOCA 1987 1113.4

$$V := Z \cdot I \cdot K \cdot C \cdot S \cdot W$$

Dead Load

Wal Load $DL_w := 20 \text{ psf}$ Consist of
CFS wall (8psf) +
1.25" CMU Cladding(12psf)

Diaphragm 1:

Floor Dead Load $DL_1 := 90 \text{ psf}$ See Building material determination

Floor Area $FloorA_1 := 9860 \text{ sf}$

Building Perimeter $P_1 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$

Wal Height $H_1 := \frac{(Height2 + Height1)}{2} = 13.75 \text{ ft}$

$$W_1 := (DL_1 \cdot FloorA_1 + P_1 \cdot H_1 \cdot DL_w) = 1.025 \times 10^6 \text{ lb}$$

Diaphragm 2:

Floor Dead Load $DL_2 := 90 \text{ psf}$

Floor Area $FloorA_2 := 10100 \text{ sf}$

Building Perimeter $P_2 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$

Wal Height $H_2 := \frac{(Height2 + Height3)}{2} = 14.125 \text{ ft}$

$$W_2 := DL_2 \cdot FloorA_2 + P_2 \cdot H_2 \cdot DL_w = 1.051 \times 10^6 \text{ lb}$$

Diaphragm 3(ROOF):

Floor Dead Load $DL_3 := 20 \text{ psf}$

Floor Area $FloorA_3 := 9560 \text{ sf}$

Building Perimeter $P_3 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$

Wal Height $H_3 := \frac{(Height3)}{2} = 7.25 \text{ ft}$

$$W_3 := DL_3 \cdot FloorA_3 + P_3 \cdot H_3 \cdot DL_w = 2.639 \times 10^5 \text{ lb}$$

$$W := W_1 + W_2 + W_3 = 2.34 \times 10^6 \text{ lb}$$

N-S:

Brace Frames are used as Lateral Force-Resisting System.

$I_w := 1.25$	Table 1113.1
$Z := \frac{3}{8}$	For Location at Zone 2, Per Table 1113.4.1
$K_w := 1.0$	For Brace Frame as LFRS, Per Table 1113.4.3
$h_n := 42.25 \quad \text{ft}$	Building Height
$D_s := 39.3 \quad \text{ft}$	Shearwall/Brace Frame dimension parallel to applied force
$T_w := 0.05 \cdot \frac{h_n}{\sqrt{D_s}} = 0.337$	Fundamental period, Per 1113.4.5.2, Eq 2
$C_w := \min\left(\frac{1}{15 \cdot \sqrt{T}}, 0.12\right) = 0.115$	Numerical coefficient Per 1113.4.4 C factor CSmax = 0.14 Per 1113.4.4
$S_w := 1.5$	PER 1113.4.6, assume soil profile=S3
$CS := \min(C \cdot S, 0.14) = 0.14$	Per 1113.4.4 C factor CS limit
$V_{\text{seismicNS}} := Z \cdot K \cdot CS \cdot I \cdot W = 1.536 \times 10^5 \quad \text{lb}$	

E-W:

Ordinary Moment Frames are used as Lateral Force-Resisting System.

$I_w := 1.25$	Table 1113.1
$Z_w := \frac{3}{8}$	For Location at Zone 2, Per Table 1113.4.1
$K_w := 1.0$	For Moment Frame as LFRS, Per Table 1113.4.3
$h_n := 42.25 \quad \text{ft}$	Building Height
$D := 147.5 \quad \text{ft}$	Building dimension parallel to applied force
$C_T := 0.035$	For Steel Moment Frame, Per 1113.4.5.2 Eq 3
$T_w := C_T \cdot h_n^{(3+4)} = 0.58$	Fundamental period, Per 1113.4.5.2, Eq 3
$C_w := \min\left(\frac{1}{15 \cdot \sqrt{T}}, 0.12\right) = 0.088$	Numerical coefficient Per 1113.4.4 C factor CSmax = 0.14 Per 1113.4.4
$S_w := 1.5$	PER 1113.4.6, assume soil profile=S3
$CS_w := \min(C \cdot S, 0.14) = 0.131$	Per 1113.4.4 C factor CS limit
$V_{\text{seismicEW}} := Z \cdot K \cdot CS \cdot I \cdot W = 1.44 \times 10^5 \quad \text{lb}$	

Wind Load

Wind load in this calculation determined by using BOCA 1987

Basic on the Map [Figure 1112.3.2]

St.Louis has a basic wind speed in miles per hour=70 mph

Structural Height=43 ft

From Table 1112.3.3a, the effective velocity pressures for this building =

WindPressure := 13 psf

Wal Pressure Coefficient Cp:

From Table 1112.2a

Windward wall = 0.8

Leeward wall = -0.5 When ($0 < L/B < 1$)

= -0.3 When ($1 < L/B < 2$)

= -0.2 When ($4 < L/B$)

Side walls = 0.7

Importance Factor: I := 1.07

Floor Height:

Height3 := 14.5 ft

Height2 := 13.75 ft

Height1 := 13.75 ft

Table 1112.2a(1)
IMPORTANCE FACTOR, I [WIND LOADS]

Category ^a	Importance factor, I ^b	
	100 miles (161 km) from hurricane oceanline, and in other areas	At hurricane oceanline ^c
I	1.00	1.05
II	1.07	1.11
III	1.07	1.11
IV	0.95	1.00

Note a. For building and structure classification categories, see Table 1112.2b.

Note b. For regions between the hurricane oceanline and 100 miles (161 km) inland the importance factor I shall be determined by linear interpolation.

Note c. Hurricane oceanlines are the Atlantic and Gulf of Mexico coastal areas.

Figure 1112.3.2
BASIC WIND SPEED (miles per hour)^a

Table 1112.3.3a
EFFECTIVE VELOCITY PRESSURES P_e (lb/ft²) FOR
BUILDINGS AND STRUCTURES (EXPOSURE B)^a

Height above grade (ft)	Basic wind speed (mph)				
	70	80	90	100	110
0-20	9	12	15	18	22
20-40	10	13	17	21	25
40-60	13	16	21	26	31
60-100	14	18	23	28	34
100-150	16	21	27	33	40
150-200	18	23	29	36	43
200-300	20	26	33	41	50
300-400	23	30	37	46	56
> 400	Per ANSI A58.1 listed in Appendix A.				

Note a. 1 pound per square foot = 47.88 Pa; 1 mile per hour = 0.447 m/s; 1 foot = 304.8 mm.

N-S:

$$\text{Height}_{\text{Total}} := \text{Height3} + \text{Height2} + \frac{\text{Height1}}{2} = 35.125 \quad \text{ft}$$

$$\text{Width3}_{\text{NS}} := 142 \quad \text{ft}$$

$$\text{Width2}_{\text{NS}} := 142$$

$$\text{Width1}_{\text{NS}} := 142$$

$$\frac{\text{Height}_{\text{Total}}}{\text{Width1}_{\text{NS}}} = 0.247 \quad < 2.5, \text{ Windward } C_p = 0.8 \text{ Leeward } C_p = 0.5$$

$$C_{p_w} := 0.8$$

$$C_{p_L} := 0.5$$

Roof Diaphragm:

$$\text{WindwardAreaRoof}_{\text{NS}} := \frac{\text{Height3}}{2} \cdot \text{Width3}_{\text{NS}} = 1.03 \times 10^3 \quad \text{sf}$$

$$\text{WindwardLoadRoof}_{\text{NS}} := \text{WindwardAreaRoof}_{\text{NS}} \cdot C_{p_w} \cdot I^2 \cdot \text{WindPressure} = 1.23 \times 10^4 \quad \text{lb}$$

$$\text{LeewardAreaRoof}_{\text{NS}} := \frac{\text{Height3}}{2} \cdot \text{Width3}_{\text{NS}} = 1.03 \times 10^3 \quad \text{sf}$$

$$\text{LeewardLoadRoof}_{\text{NS}} := \text{LeewardAreaRoof}_{\text{NS}} \cdot C_{p_L} \cdot I^2 \cdot \text{WindPressure} = 7.66 \times 10^3 \quad \text{lb}$$

$$\text{WindLoadRoof}_{\text{NS}} := \text{WindwardLoadRoof}_{\text{NS}} + \text{LeewardAreaRoof}_{\text{NS}} = 1.329 \times 10^4 \quad \text{lb}$$

3rd Floor Diaphragm:

$$\text{WindwardArea3rd}_{\text{NS}} := \left(\frac{\text{Height3}}{2} + \frac{\text{Height2}}{2} \right) \cdot \text{Width2}_{\text{NS}} = 2.006 \times 10^3 \quad \text{sf}$$

$$\text{WindwardLoad3rd}_{\text{NS}} := \text{WindwardArea3rd}_{\text{NS}} \cdot C_{p_w} \cdot I^2 \cdot \text{WindPressure} = 2.39 \times 10^4 \quad \text{lb}$$

$$\text{LeewardArea3rd}_{\text{NS}} := \left(\frac{\text{Height3}}{2} + \frac{\text{Height2}}{2} \right) \cdot \text{Width2}_{\text{NS}} = 2.006 \times 10^3 \quad \text{sf}$$

$$\text{LeewardLoad3rd}_{\text{NS}} := \text{LeewardArea3rd}_{\text{NS}} \cdot C_{p_L} \cdot I^2 \cdot \text{WindPressure} = 1.49 \times 10^4 \quad \text{lb}$$

$$\text{WindLoad3rd}_{\text{NS}} := \text{WindwardLoad3rd}_{\text{NS}} + \text{LeewardLoad3rd}_{\text{NS}} = 3.881 \times 10^4 \quad \text{lb}$$

2nd Floor Diaphragm:

$$\text{WindwardArea2nd}_{\text{NS}} := \left(\frac{\text{Height2}}{2} + \frac{\text{Height1}}{2} \right) \cdot \text{Width1}_{\text{NS}} = 1.952 \times 10^3 \quad \text{sf}$$

$$\text{WindwardLoad2nd}_{\text{NS}} := \text{WindwardArea2nd}_{\text{NS}} \cdot C_{p_w} \cdot I^2 \cdot \text{WindPressure} = 2.32 \times 10^4 \quad \text{lb}$$

$$\text{LeewardArea2nd}_{\text{NS}} := \left(\frac{\text{Height2}}{2} + \frac{\text{Height1}}{2} \right) \cdot \text{Width1}_{\text{NS}} = 1.952 \times 10^3 \quad \text{sf}$$

$$\text{LeewardLoad2nd}_{\text{NS}} := \text{LeewardArea2nd}_{\text{NS}} \cdot C_{p_L} \cdot I^2 \cdot \text{WindPressure} = 1.45 \times 10^4 \quad \text{lb}$$

$$\text{WindLoad2nd}_{\text{NS}} := \text{WindwardLoad2nd}_{\text{NS}} + \text{LeewardLoad2nd}_{\text{NS}} = 3.778 \times 10^4 \quad \text{lb}$$

BASE SHEAR FROM WIND

$$\text{WindLoadBase}_{\text{NS}} := \text{WindLoadRoof}_{\text{NS}} + \text{WindLoad3rd}_{\text{NS}} + \text{WindLoad2nd}_{\text{NS}} = 8.988 \times 10^4 \quad \text{lb}$$

E-W

$$\text{Height}_{\text{Total}} = 35.125 \quad \text{ft}$$

$$\text{Width}_{3\text{EW}} := 70 \quad \text{ft}$$

$$\text{Width}_{2\text{EW}} := 70 \quad \text{ft}$$

$$\text{Width}_{1\text{EW}} := 76 \quad \text{ft}$$

$$\frac{\text{Height}_{\text{Total}}}{\text{Width}_{3\text{EW}}} = 0.502 \quad < 2.5, \text{ Windward } C_p = 0.8 \text{ Leeward } C_p = 0.5$$

$$C_{p_w} := 0.8$$

$$C_{p_L} := 0.5$$

Roof Diaphragm:

$$\text{WindwardAreaRoof}_{\text{EW}} := \frac{\text{Height}_3}{2} \cdot \text{Width}_{3\text{EW}} = 507.5 \quad \text{sf}$$

$$\text{WindwardLoadRoof}_{\text{EW}} := \text{WindwardAreaRoof}_{\text{EW}} \cdot C_{p_w} \cdot I^2 \cdot \text{WindPressure} = 6.04 \times 10^3 \quad \text{lb}$$

$$\text{LeewardAreaRoof}_{\text{EW}} := \frac{\text{Height}_3}{2} \cdot \text{Width}_{3\text{EW}} = 507.5 \quad \text{sf}$$

$$\text{LeewardLoadRoof}_{\text{EW}} := \text{LeewardAreaRoof}_{\text{EW}} \cdot C_{p_L} \cdot I^2 \cdot \text{WindPressure} = 3.78 \times 10^3 \quad \text{lb}$$

$$\text{WindLoadRoof}_{\text{EW}} := \text{WindwardLoadRoof}_{\text{EW}} + \text{LeewardLoadRoof}_{\text{EW}} = 6.55 \times 10^3 \quad \text{lb}$$

3rd Floor Diaphragm:

$$\text{WindwardArea3rd}_{EW} := \left(\frac{\text{Height3}}{2} + \frac{\text{Height2}}{2} \right) \cdot \text{Width2}_{EW} = 988.75 \quad \text{sf}$$

$$\text{WindwardLoad3rd}_{EW} := \text{WindwardArea3rd}_{EW} \cdot C_{p_w} \cdot I^2 \cdot \text{WindPressure} = 1.18 \times 10^4 \quad \text{lb}$$

$$\text{LeewardArea3rd}_{EW} := \left(\frac{\text{Height3}}{2} + \frac{\text{Height2}}{2} \right) \cdot \text{Width2}_{EW} = 988.75 \quad \text{sf}$$

$$\text{LeewardLoad3rd}_{EW} := \text{LeewardArea3rd}_{EW} \cdot C_{p_L} \cdot I^2 \cdot \text{WindPressure} = 7.36 \times 10^3 \quad \text{lb}$$

$$\text{WindLoad3rd}_{EW} := \text{WindwardLoad3rd}_{EW} + \text{LeewardLoad3rd}_{EW} = 1.913 \times 10^4 \quad \text{lb}$$

2nd Floor Diaphragm:

$$\text{WindwardArea2nd}_{EW} := \left(\frac{\text{Height2}}{2} + \frac{\text{Height1}}{2} \right) \cdot \text{Width1}_{NS} = 1.952 \times 10^3 \quad \text{sf}$$

$$\text{WindwardLoad2nd}_{EW} := \text{WindwardArea2nd}_{NS} \cdot C_{p_w} \cdot I^2 \cdot \text{WindPressure} = 2.32 \times 10^4 \quad \text{lb}$$

$$\text{LeewardArea2nd}_{EW} := \left(\frac{\text{Height2}}{2} + \frac{\text{Height1}}{2} \right) \cdot \text{Width1}_{NS} = 1.952 \times 10^3 \quad \text{sf}$$

$$\text{LeewardLoad2nd}_{EW} := \text{LeewardArea2nd}_{EW} \cdot C_{p_L} \cdot I^2 \cdot \text{WindPressure} = 1.45 \times 10^4 \quad \text{lb}$$

$$\text{WindLoad2nd}_{EW} := \text{WindwardLoad2nd}_{EW} + \text{LeewardLoad2nd}_{EW} = 3.778 \times 10^4 \quad \text{lb}$$

BASE SHEAR FROM WIND

$$\text{WindLoadBase}_{EW} := \text{WindLoadRoof}_{EW} + \text{WindLoad3rd}_{EW} + \text{WindLoad2nd}_{EW} = 6.346 \times 10^4 \quad \text{lb}$$

Design Lateral Load

N-S(yy): Brace Frame

$$V_{\text{Wind}} := \text{WindLoadBase}_{\text{NS}} = 8.988 \times 10^4 \quad \text{lb}$$

$$V_{\text{Seismic}} := V_{\text{seismicNS}} = 1.536 \times 10^5 \quad \text{lb}$$

$$\text{ControlCheck}_{\text{NS}} = \text{"Seismic Control"}$$

$$V_{\text{NS}} := \frac{V_{\text{Seismic}}}{1000} = 153.561 \quad \text{Kips}$$

E-W(xx): Moment Frame

$$\underline{V_{\text{Wind}}} := \text{WindLoadBase}_{\text{EW}} = 6.346 \times 10^4 \quad \text{lb}$$

$$\underline{V_{\text{Seismic}}} := V_{\text{seismicEW}} = 1.44 \times 10^5 \quad \text{lb}$$

$$\text{ControlCheck}_{\text{EW}} = \text{"Seismic Control"}$$

$$V_{\text{EW}} := \frac{V_{\text{Seismic}}}{1000} = 144.023 \quad \text{Kips}$$

MOMENT FRAME DESIGN

Moment Frame Design Criteria

1. Moment-resisting Ordinary steel frames using $K=1.0$ shall have a frame conforming to the requirements for the AISC Spec listed in Appendix A
2. Drift limitation: Lateral Deflection (or drift) shall not exceed 0.005 times the story height

$$\text{Height3} := 14.5 \quad \text{ft}$$

$$\text{Height2} := 13.75 \quad \text{ft}$$

$$\text{Height1} := 13.75 \quad \text{ft}$$

Lateral Deflection Limit:

$$\text{Drift1} := 0.005 \cdot \text{Height1} \cdot 12 \cdot \frac{K}{I} = 0.66 \quad \text{inches}$$

$$\text{Drift2} := 0.005 \cdot \text{Height2} \cdot 12 \cdot \frac{K}{I} = 0.66 \quad \text{inches}$$

$$\text{Drift3} := 0.005 \cdot \text{Height3} \cdot 12 \cdot \frac{K}{I} = 0.696 \quad \text{inches}$$

Roof Diaphragm:

Flexible Diaphragm, force distributed based on their tributary area (mass).

$$\text{Area1} := 1652 \quad \text{sf}$$

$$\text{Area2} := 5003 \quad \text{sf}$$

$$\text{Area3} := 3240 \quad \text{sf}$$

Total Lateral Force at Roof (E-W):

$$\text{TotalLateralForce} := 30 \quad \text{Kips}$$

Force distribution at the Roof:

$$F_{\text{Roof_MF1}} := \text{TotalLateralForce} \cdot \frac{\text{Area1}}{\text{Area1} + \text{Area2} + \text{Area3}} = 5.009 \quad \text{Kips}$$

$$F_{\text{Roof_MF2}} := \text{TotalLateralForce} \cdot \frac{\text{Area2}}{\text{Area1} + \text{Area2} + \text{Area3}} = 15.168 \quad \text{Kips}$$

$$F_{\text{Roof_MF3}} := \text{TotalLateralForce} \cdot \frac{\text{Area3}}{\text{Area1} + \text{Area2} + \text{Area3}} = 9.823 \quad \text{Kips}$$

Stiffness R:

Start with assuming the stiffness will be similar to each other at first trial,

Input the location and stiffness of each frames(include momentframe and braceframe) to Rigid Diaphragm distribution third/second floor Spreadsheet:

ThirdFloor Diaphragm:

Moment Frame

MF1_Third := 25.33 kips

MF2_Third := 25.63 kips

MF3_Third := 26.04 kips

Similarly at Second Floor

Moment Frame

MF1_Second := 12.4 kips

MF2_Second := 12.5 kips

MF3_Second := 12.63 kips

Moment Frame Design:

MF1:

Load at different Diaphragm:

$$\begin{aligned} F_{\text{Roof_MF1}} &= 5.009 \\ \text{MF1_Third} &= 25.33 \\ \text{MF1_Second} &= 12.4 \end{aligned} \quad \text{Kips}$$

Roof Dead Load:

$$\begin{aligned} \text{Dead Load} &= 20 \text{ psf} \\ \text{Trib Width} &= 14.33 \text{ ft} \\ 20 \cdot \frac{14.333}{1000} &= 0.287 \text{ klf} \end{aligned}$$

$$\begin{aligned} \text{Live Load} &= 20 \text{ psf} \\ \text{Trib Width} &= 14.33 \text{ ft} \\ 20 \cdot \frac{14.333}{1000} &= 0.287 \text{ klf} \end{aligned}$$

Floor Dead Load:

$$\begin{aligned} \text{Dead Load} &= 90 \text{ psf} \\ \text{Trib Width} &= 14.33 \text{ ft} \\ 90 \cdot \frac{14.333}{1000} &= 1.29 \text{ klf} \end{aligned}$$

$$\begin{aligned} \text{Live Load} &= 50 \text{ psf} \\ \text{Trib Width} &= 14.33 \text{ ft} \\ 50 \cdot \frac{14.333}{1000} &= 0.717 \text{ klf} \end{aligned}$$

Drift Limit:

$$\begin{aligned} \text{Drift1} &= 0.66 & \text{Deflection1} &:= \text{Drift1} = 0.66 \\ \text{Drift2} &= 0.66 & \text{Deflection2} &:= \text{Deflection1} + \text{Drift2} = 1.32 \\ \text{Drift3} &= 0.696 & \text{Deflection3} &:= \text{Deflection2} + \text{Drift3} = 2.016 \end{aligned}$$

Check real stiffness at Frame 1

1 Kips of Unit Load at Second Floor

$$\begin{aligned} E1_2 &:= 1 \text{ kips} \\ \Delta1_2 &:= 0.0137 \text{ in} \\ \text{MF1_R_2} &:= \frac{E1_2}{\Delta1_2} = 72.99 \frac{\text{Kips}}{\text{in}} \end{aligned}$$

1 Kips of Unit Load at Third Floor

$$\begin{aligned} E1_3 &:= 1 \text{ kips} \\ \Delta1_3 &:= 0.0232 \text{ in} \\ \text{MF1_R_3} &:= \frac{E1_3}{\Delta1_3} = 43.10 \frac{\text{Kips}}{\text{in}} \end{aligned}$$

MF2:

Load at different Diaphragm:

$$\begin{aligned}
 F_{\text{Roof_MF2}} &= 15.168 \\
 MF2_{\text{Third}} &= 25.63 \\
 MF2_{\text{Second}} &= 12.5
 \end{aligned}
 \quad \text{Kips}$$

Roof Dead Load:

$$\begin{aligned}
 &\text{Dead Load} \quad 20 \text{ psf} \\
 &\text{Trib Width} = 19 \quad \text{ft} \\
 &20 \cdot \frac{19}{1000} = 0.38 \quad \text{klf}
 \end{aligned}$$

$$\begin{aligned}
 &\text{Live Load} \quad 20 \text{ psf} \\
 &\text{Trib Width} = 19 \quad \text{ft} \\
 &20 \cdot \frac{19}{1000} = 0.38 \quad \text{klf}
 \end{aligned}$$

Floor Dead Load:

$$\begin{aligned}
 &\text{Dead Load} \quad 90 \text{ psf} \\
 &\text{Trib Width} = 19 \quad \text{ft} \\
 &90 \cdot \frac{19}{1000} = 1.71 \quad \text{klf}
 \end{aligned}$$

$$\begin{aligned}
 &\text{Live Load} \quad 50 \text{ psf} \\
 &\text{Trib Width} = 19 \quad \text{ft} \\
 &50 \cdot \frac{19}{1000} = 0.95 \quad \text{klf}
 \end{aligned}$$

Drift Limit:

$$\begin{aligned}
 \text{Drift1} &= 0.66 & \text{Deflection1} &= 0.66 \\
 \text{Drift2} &= 0.66 & \text{Deflection2} &= 1.32 \\
 \text{Drift3} &= 0.696 & \text{Deflection3} &= 2.016
 \end{aligned}$$

Check real stiffness at Frame 2

1 Kips of Unit Load at Second Floor

$$\begin{aligned}
 E2_2 &:= 1 \text{ kips} \\
 \Delta2_2 &:= 0.011 \text{ in} \\
 MF2_R_2 &:= \frac{E2_2}{\Delta2_2} = 90.9 \frac{\text{Kips}}{\text{in}}
 \end{aligned}$$

1 Kips of Unit Load at Third Floor

$$\begin{aligned}
 E2_3 &:= 1 \text{ kips} \\
 \Delta2_3 &:= 0.0181 \text{ in} \\
 MF2_R_3 &:= \frac{E2_3}{\Delta2_3} = 55.24 \frac{\text{Kips}}{\text{in}}
 \end{aligned}$$

MF3:

Load at different Diaphragm:

$$F_{\text{Roof_MF3}} = 9.823$$

$$MF3_{\text{Third}} = 26.04 \quad \text{Kips}$$

$$MF3_{\text{Second}} = 12.63$$

Roof Dead Load:

Dead Load 20 psf

Trib Width = 14.65 ft

$$20 \cdot \frac{14.65}{1000} = 0.293 \quad \text{klf}$$

Live Load 20 psf

Trib Width = 14.65 ft

$$20 \cdot \frac{14.65}{1000} = 0.293 \quad \text{klf}$$

Floor Dead Load:

Dead Load 90 psf

Trib Width = 14.65 ft

$$90 \cdot \frac{14.65}{1000} = 1.319 \quad \text{klf}$$

Live Load 50 psf

Trib Width = 14.65 ft

$$50 \cdot \frac{14.65}{1000} = 0.733 \quad \text{klf}$$

Drift Limit:

$$\text{Drift1} = 0.66$$

$$\text{Drift2} = 0.66$$

$$\text{Drift3} = 0.696$$

$$\text{Deflection1} = 0.66$$

$$\text{Deflection2} = 1.32$$

$$\text{Deflection3} = 2.016$$

Check real stiffness at Frame 3

1 Kips of Unit Load at Second Floor

$$E3_2 := 1 \quad \text{kips}$$

$$\Delta3_2 := 0.0116 \quad \text{in}$$

$$MF3_R_2 := \frac{E3_2}{\Delta3_2} = 86.21 \frac{\text{Kips}}{\text{in}}$$

1 Kips of Unit Load at Third Floor

$$E3_3 := 1 \quad \text{kips}$$

$$\Delta3_3 := 0.0201 \quad \text{in}$$

$$MF3_R_3 := \frac{E3_3}{\Delta3_3} = 49.751 \frac{\text{Kips}}{\text{in}}$$

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Structural Solutions

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$F_x =$	37.5 k	$A_x =$	1.00	
$L_y =$	67 ft	$M_t =$	-2 k-ft	
$CF_y =$	34 ft	$M_{ta} =$	126 k-ft	± 5%
$CR_v =$	33.55 ft	$\Sigma R(x_{cr})^2 + \Sigma R(y_{cr})^2 =$	1.272E+06	

Input the location and **Update** stiffness of each frames(include momentframe and braceframe) to Rigid Diaphragm distribution third floor Spreadsheet:

Second Floor

MF1_R_2 = 72.993 k/in

MF2_R_2 = 90.909 k/in

MF3_R_2 = 86.207 k/in

Third Floor

MF1_R_3 = 43.103 k/in

MF2_R_3 = 55.249 k/in

MF3_R_3 = 49.751 k/in

From Updated Spreadsheet, force distribution at
Second Floor

Moment Frame

MF1 := 11.2 kips

MF2 := 13.67 kips

MF3 := 13.22 kips

Third Floor

Moment Frame

MF1 := 22.07 kips

MF2 := 28.66 kips

MF3 := 26.27 kips

Update Load at Each Frame:

MF1 ROOF := 5 Kips

THIRD := 22.07 Kips

SECOND := 11.2 Kips

MF2 ROOF := 15.2 Kips

THIRD := 28.66 Kips

SECOND := 13.67 Kips

MF3 ROOF := 9.8 Kips

THIRD := 26.27 Kips

SECOND := 13.22 Kips

Checked, Current design meet drift and strength requirement for updated force, see sample calculation

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in ²]	Iyy [in ⁴]	Izz [in ⁴]	J [in ⁴]
1	MF-BEA...	W24x104...	Beam	Wide Flan...	A36 Gr.36	Typical	30.6	259	3100	4.72
2	MF-BEA...	W21x62 ...	Beam	Wide Flan...	A36 Gr.36	Typical	18.3	57.5	1330	1.83
3	MF-BEA...	W16x45 ...	Beam	Wide Flan...	A36 Gr.36	Typical	13.3	32.8	586	1.11
4	BF-COLU...	W18x106...	Column	Wide Flan...	A36 Gr.36	Typical	31.1	220	1910	7.48

Node Loads and Enforced Displacements (BLC 3 : E)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (... Inactive [(k, k-ft), (in,...
1	N2	L	X	5.1 Active
2	N5	L	X	25.33 Active
3	N7	L	X	12.4 Active

Story Drift X-Service

available free of	Story (El...		Story Dri...	Loc	LC	Drift Rati...	Loc	LC	2nd/1st...	Loc	LC	
	1	Diaph.: 1...	max	0.1431	0, 41	3	0.0837	0, 41	6	1.0079	0, 41	5
	2		min	-0.1457	0, 41	6	0.0005	0, 41	1	1	0, 41	1
	3	Diaph.: 2...	max	0.2634	0, 0	5	0.1643	0, 41	6	1.0106	0, 0	5
	4		min	-0.2662	0, 41	6	0.0004	0, 41	1	1	0, 41	1
	5	Diaph.: 3...	max	0.6645	0, 0	5	0.3966	0, 0	6	1.0175	0, 0	6
	6		min	-0.6662	0, 0	6	0	0, 41	1	1	0, 41	1

Story Drift Z-Service

	Story (El...	Story Dri...	Loc	LC	Drift Rati...	Loc	LC	2nd/1st...	Loc	LC
1	Diaph.: 1...	max			NC			NC		
2		min			NC			NC		
3	Diaph.: 2...	max			NC			NC		
4		min			NC			NC		
5	Diaph.: 3...	max			NC			NC		
6		min			NC			NC		

Story Drift X-Strength

No Data to Print...

Story Drift Z-Strength

No Data to Print...

Member Distributed Loads (BLC 1 : DL)

	Member Label	Direction	Start Magnitud...	End Magnitude...	Start Location [...	End Location [(...	Inactive [(k, k-f...
1	M4	Y	-0.287	-0.287	0	%100	Active
2	M5	Y	-0.287	-0.287	0	%100	Active
3	M8	Y	-1.29	-1.29	0	%100	Active
4	M9	Y	-1.29	-1.29	0	%100	Active
5	M6	Y	-1.29	-1.29	0	%100	Active
6	M7	Y	-1.29	-0.86	0	%100	Active

Member Distributed Loads (BLC 2 : LL)

	Member Label	Direction	Start Magnitud...	End Magnitude...	Start Location [...	End Location [(...	Inactive [(k, k-f...
1	M4	Y	-0.287	-0.287	0	%100	Active
2	M5	Y	-0.287	-0.287	0	%100	Active
3	M8	Y	-0.72	-0.72	0	%100	Active
4	M9	Y	-0.72	-0.72	0	%100	Active
5	M6	Y	-0.72	-0.72	0	%100	Active
6	M7	Y	-0.72	-0.72	0	%100	Active

Maximum Member Section Forces

	Me...		Axi...	Loc...	LC	y S...	Loc...	LC	z S...	Loc...	LC	Tor...	Loc...	LC	y-y...	Loc...	LC	z-z...	Loc...	LC
1	M1	max	78....	0	6	12....	14	3	0	42	6	0	42	6	0	42	6	204....	14	6
2		min	1.2....	13....	3	-14....	0	6	0	0	1	0	0	1	0	0	1	-17....	14	3
3	M2	max	110....	0	5	16....	14	3	0	42	6	0	42	6	0	42	6	243....	14	6
4		min	6.3....	42	4	-17....	0	6	0	0	1	0	0	1	0	0	1	-23....	14	3
5	M3	max	76....	0	5	14....	14	5	0	42	6	0	42	6	0	42	6	169....	14	4
6		min	2.2....	42	4	-12....	0	4	0	0	1	0	0	1	0	0	1	-20....	14	5
7	M4	max	0	19	6	7.7....	0	6	0	19	6	0	19	6	0	19	6	36....	0	6
8		min	0	0	1	-7.6....	19	5	0	0	1	0	0	1	0	0	1	-12....	6.7....	5
9	M5	max	0	22	6	8.2....	0	6	0	22	6	0	22	6	0	22	6	40....	22	5
10		min	0	0	1	-8.2....	22	5	0	0	1	0	0	1	0	0	1	-15....	13....	6
11	M6	max	0	19	6	39....	0	6	0	19	6	0	19	6	0	19	6	269....	0	6
12		min	0	0	1	-42....	19	5	0	0	1	0	0	1	0	0	1	-19....	0	3
13	M7	max	0	22	6	39....	0	6	0	22	6	0	22	6	0	22	6	265....	22	5
14		min	0	0	1	-36....	22	5	0	0	1	0	0	1	0	0	1	-17....	22	4
15	M8	max	0	19	5	26....	0	6	0	19	6	0	19	6	0	19	6	129....	19	5
16		min	0	0	4	-27....	19	5	0	0	1	0	0	1	0	0	1	-48....	5.9....	5
17	M9	max	0	22	6	28....	0	6	0	22	6	0	22	6	0	22	6	142....	0	6
18		min	0	0	1	-27....	22	5	0	0	1	0	0	1	0	0	1	-56....	13.75	6

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SAMPLE BEAM CALCULATION

AISC Eighth Edition Column Design Wide Flange Member Only		Project ID:	19051
			Project Name:	ATC
			Engineer:	HL
			Date:	4/28/2021

Member # =	MF1-B1	Shape	W 24x104
------------	--------	-------	----------

Properties	Geometry	
	A	30.6 in. ²
	d	24.06 in.
	b _f	12.75 in.
	t _f	0.75 in.
	t _w	0.5 in.
	W(weight)	104 #/ft.
	r _T	3.35
	d/A _T	2.52

Compact Section Criteria	
b _f /2t _f	8.5
F _y	58.5 ksi
d/t _w	48.1
F _y	— ksi
F _y	28.5 ksi

Elastic Properties					
X-X			Y-Y		
I _x	3100	in. ⁴	I _y	259	in. ⁴
S _x	258	in. ³	S _y	40.7	in. ³
r _x	10.1	in.	r _y	2.91	in.
Plastic Modulus					
Z _x	289	in. ³	Z _y	62.4	in. ³

Torsional Constant	
J	4.72 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Eighth Edition

Building info:

Steel Properties

Total Member Length =	22	ft.
Beam Spacing (w) =	15	ft.
Tributary Area to Col=	0	ft.^2
K=	1	
Unbraced Length =	12	ft.

Loading

Dead Load (D)=	90	psf
Live Load(L)=	50	psf
Roof Live Load(Lr)=	0	psf
Snow Load(S)=	25	psf

Excentricity on Column

ex =	2	in.
ey =	0	in.

Deflection Criteria

Total Load	L/360=	0.73	in.
------------	--------	------	-----

Max Moment About Major Axis:

MD(Max Dead)=	980.1	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	980.1	kips-in
ML(Max Live)=	544.5	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	544.5	kips-in
M(Lr or S)(Max Live)=	272.25	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	272.25	kips-in
Mu=	3240	kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Shear

VD(Max Dead)=	14.85	kips
VD(Dead Overwrite)=	0	kips
VMD(Max Use)=	14.85	kips
VL(Max Live)=	8.25	kips
VL(Live Overwrite)=	0	kips
VL(Max Use)=	8.25	kips
V(Lr or S)(Max Live)=	6.27	kips
V(Lr or S)(Live Overwrite)=	0	kips
V(Lr or S)(Max Use)=	6.27	kips
Vu=	45	kips

Max Axial Load

PD(Max Dead)=	0	kips
PD(Dead Overwrite)=	0.1	kips
PD(Max Use)=	0.1	kips
PL(Max Live)=	0	kips
PL(Live Overwrite)=	0.1	kips
PL(Max Use)=	0.1	kips
P(Lr or S)(Max Live)=	0	kips
P(Lr or S)(Live Overwrite)=	0.1	kips
P(Lr or S)(Max Use)=	0.1	kips
P(E or W)=	1	kips
Pu=	1	kips

Solution Summary:

Try **W 24x104** <---- Change W shapes

Member	Major Axis Beam Bending	40%
	Beam Shear	26%
	Minor Axis Beam Bending	0%
	Axial	0%
	Deflection	21%
	Combined Bending and Axial	40%
Overall Check		40%

Column Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Column Height= 22 ft
K= 1
KL= 22 ft
KL/r= 90.7

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	126
if $\frac{KL}{r} \leq C_c$ then $F_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
if $\frac{KL}{r} \geq C_c$ then $F_a = \frac{12\pi^2 E}{23 (\frac{KL}{r})^2}$	Section 1.5.1.3.2
Column in Tension	
$F_a = 0.6F_y$	Section 1.5.1.1

Fa = 14.1 ksi

Calculate Stresses in Column

Axial Stress: fa= Pu/A= 0 ksi Okay Increases by 1/3

Non-Composite Major Axis Beam Design AISC Eighth Edition

Unbraced length (Lb)= 12 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 65 / \sqrt{F_y}$	8.5	10.8	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	48.1	68.7	Good
$l_b \leq 76 b_f / \sqrt{F_y}$	144	161.5	Good
$l_b \leq \frac{20,000}{(d / A_f) F_y}$	144	220.8	Good
Use Typical Bending Stress Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	8.5	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	8.5	15.8	Good
N/A			

Typical Bending Stress Equation: $F_{bx} = 0.66F_y$

Fbx= 31.6008 ksi Increases by 1/3

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

Fbx= N/A ksi Increases by 1/3

Shear Stress Equation: $F_v = 0.40F_y$

Fv= 14.4 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.75F_y$

Fby= 27 ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	12.6	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w*d) =$	3.7	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5wL^4}{384EI} - C =$	0.15	in.	Okay

Check Combined Stressed

$C_{mx} =$	1
$C_{my} =$	1

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{C_{mx}f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{C_{my}f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.4	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.4	Less than 1.0, Okay

SAMPLE COLUMN CALCULATION

AISC Eighth Edition Column Design Wide Flange Member Only

Project ID:	19051
Project Name:	ATC
Engineer:	HL
Date:	11/20/2020

Member # = **MF1-C1**

Shape **W 18x106**

Properties	Geometry	
	A	31.1 <i>in.²</i>
	d	18.73 <i>in.</i>
	b_f	11.2 <i>in.</i>
	t_f	0.94 <i>in.</i>
	t_w	0.59 <i>in.</i>
	W(weight)	106 <i>#/ft.</i>
	r_T	3
	d/A_f	1.78

Compact Section Criteria	
b_f/2t_f	6
F_y	— <i>ksi</i>
d/t_w	31.7
F_y	— <i>ksi</i>
F_y	— <i>ksi</i>

Elastic Properties					
X-X			Y-Y		
I_x	1910	<i>in.⁴</i>	I_y	220	<i>in.⁴</i>
S_x	204	<i>in.³</i>	S_y	39.4	<i>in.³</i>
r_x	7.84	<i>in.</i>	r_y	2.66	<i>in.</i>
Plastic Modulus					
Z_x	230	<i>in.³</i>	Z_y	60.5	<i>in.³</i>

Torsional Constant	
J	7.48 <i>in.⁴</i>

Warping Constant	
C_w	— <i>in.⁶</i>

Material Property	
E	29000 <i>ksi</i>
F_y	36 <i>ksi</i>

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Non-Composite Beam/Column Design AISC Eighth Edition

Building info:

Steel Properties

Total Member Length =	14.5	ft.
Beam Spacing (w) =	0	ft.
Tributary Area to Col=	0	ft.^2
K=	1	
Unbraced Length =	14.5	ft.

Loading

Dead Load (D)=	90	psf
Live Load(L)=	40	psf
Roof Live Load(Lr)=	0	psf
Snow Load(S)=	25	psf

Excentricity on Column

ex =	2	in.
ey =	0	in.

Deflection Criteria

Total Load	L/360=	0.48	in.
------------	--------	------	-----

Max Moment About Major Axis:

MD(Max Dead)=	0.2	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0.2	kips-in
ML(Max Live)=	0.2	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0.2	kips-in
M(Lr or S)(Max Live)=	0.2	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0.2	kips-in
Mu=	2940	kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Shear

VD(Max Dead)=	0	kips
VD(Dead Overwrite)=	0	kips
VMD(Max Use)=	0	kips
VL(Max Live)=	0	kips
VL(Live Overwrite)=	0	kips
VL(Max Use)=	0	kips
V(Lr or S)(Max Live)=	0	kips
V(Lr or S)(Live Overwrite)=	0	kips
V(Lr or S)(Max Use)=	0	kips
Vu=	18	kips

Max Axial Load

PD(Max Dead)=	0	kips
PD(Dead Overwrite)=	0.1	kips
PD(Max Use)=	0.1	kips
PL(Max Live)=	0	kips
PL(Live Overwrite)=	0.1	kips
PL(Max Use)=	0.1	kips
P(Lr or S)(Max Live)=	0	kips
P(Lr or S)(Live Overwrite)=	0.1	kips
P(Lr or S)(Max Use)=	0.1	kips
P(E or W)=	112	kips
Pu=	112	kips

Solution Summary:

Try **W 18x106** <---- Change W shapes

Member	Major Axis Beam Bending	45%
	Beam Shear	11%
	Minor Axis Beam Bending	0%
	Axial	21%
	Deflection	0%
	Combined Bending and Axial	67%
Overall Check		67%

DEFLECTION DID NOT APPLY TO BM-COL. CHECK IN THIS CASE. DRIFT HAS BEEN CHECKED IN PREVIOUS CALC.

Column Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Column Height= 14.5 ft
K= 1
KL= 14.5 ft
KL/r= 65.4

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	126
if $\frac{KL}{r} \leq C_c$ then $F_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 15.1.3.1
if $\frac{KL}{r} \geq C_c$ then $F_a = \frac{12\pi^2 E}{23 (\frac{KL}{r})^2}$	Section 15.1.3.2
Column in Tension	
$F_a = 0.6F_y$	Section 15.1.1

Fa = 16.9 ksi

Calculate Stresses in Column

Axial Stress: fa= Pu/A= 3.6 ksi Okay Increases by 1/3

Non-Composite Major Axis Beam Design AISC Eighth Edition

Unbraced length (Lb)= 14.5 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 65 / \sqrt{F_y}$	6	10.8	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_u}{F_y}) / \sqrt{F_y}$	31.7	52.7	Good
$l_b \leq 76 b_f / \sqrt{F_y}$	174	141.9	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d/A_f) F_y}$	174	312.3	Good
			Use Alternate Equation

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	6	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	6	15.8	Good
			Use Alternate Bending Stress Equation

Typical Bending Stress Equation: $F_{bx} = 0.66F_y$

Fbx= N/A ksi Increases by 1/3

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

Fbx= 31.92 ksi Increases by 1/3

Shear Stress Equation: $F_v = 0.40F_y$

Fv= 14.4 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.75F_y$

Fby= 27 ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	14.4	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	1.6	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5wL^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

C _{mx} =	1
C _{my} =	1

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{C_{mx}f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{C_{my}f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.67	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.62	Less than 1.0, Okay

BRACED FRAME DESIGN

Brace Frame Design Criteria

1. All members in braced frames shall be designed for **1.25** times the lateral force determined from BOCA 1987.

1113.9.5 Braced frames: In Seismic Zones 3 and 4, and for buildings having an importance factor I greater than 1.0 and located in Seismic Zone 2, all members in braced frames shall be designed for 1.25 times the force determined in accordance with Section 1113.4. Steel members in braced frames shall be limited to those grades listed in Section 1113.9.3.3. Reinforced concrete members in braced frames shall be provided with the transverse confinement reinforcement required in Section 1113.9.3.3.

2. Drift limitation: Lateral Deflection(or drift) shall not exceed 0.005 times the story height per 1113.7 Displacement calculated from application shall be multiplied by I/K

Height3 := 14.5 ft

Height2 := 13.75 ft

Height1 := 13.75 ft

Lateral Deflection Limit:

$$\text{Drift1} := 0.005 \cdot \text{Height1} \cdot 12 \cdot \frac{K}{I} = 0.66 \text{ inches}$$

$$\text{Drift2} := 0.005 \cdot \text{Height2} \cdot 12 \cdot \frac{K}{I} = 0.66 \text{ inches}$$

$$\text{Drift3} := 0.005 \cdot \text{Height3} \cdot 12 \cdot \frac{K}{I} = 0.696 \text{ inches}$$

Roof Diaphragm:

Flexible Diaphragm, force distributed basic on their tributary area(mass).

Area1 := 3310 sf

Area2 := 3210 sf

Area3 := 3550 sf

Total Lateral Force at Roof (N-S):

TotalLateralForce := 38.4 Kips (From Vertical Distribution)

Force at OCBF 1 Roof:

$$F_{\text{Roof_OCBF1}} := \text{TotalLateralForce} \cdot \frac{\text{Area1}}{\text{Area1} + \text{Area2} + \text{Area3}} = 12.622 \text{ Kips}$$

$$F_{\text{Roof_OCBF2}} := \text{TotalLateralForce} \cdot \frac{\text{Area2}}{\text{Area1} + \text{Area2} + \text{Area3}} = 12.241 \text{ Kips}$$

$$F_{\text{Roof_OCBF3}} := \text{TotalLateralForce} \cdot \frac{\text{Area3}}{\text{Area1} + \text{Area2} + \text{Area3}} = 13.537 \text{ Kips} \quad \text{Control}$$

In N-S Direction,

3 Brace Frames are identical and they will have similar stiffness.

Stiffness R:

$$\frac{29.5}{0.108} = 273.148 \quad \frac{\text{kips}}{\text{in}}$$

Since all three braces frames are very similar, we will design with the worst case

Lateral Load at different diaphragm: (Note 25% force increase are required per 1113.9.5)

Roof:	$\text{MaxRoof_OCBF} \cdot 1.25 = 16.922$	Kips	
Third Floor:	$36.6 \cdot 1.25 = 45.75$	Kips	<-----From Spreadsheet
Second Floor:	$17.84 \cdot 1.25 = 22.3$	Kips	<-----From Spreadsheet

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in ²]	Iyy [in ⁴]	Izz [in ⁴]	J [in ⁴]
1	BF-BEAM...	W18x55	Beam	Wide Flan...	A36 Gr.36	Typical	16.2	44.9	890	1.66
2	BF-BEAM...	W16x50	Beam	Wide Flan...	A36 Gr.36	Typical	14.7	37.2	659	1.52
3	BF-COLU...	W14x61	Column	Wide Flan...	A36 Gr.36	Typical	17.9	107	640	2.2
4	BF-BRAC...	STS5x5x...	VBrace	Tube	A36 Gr.36	Typical	4.59	17.911	16.9	26.793
5	BF-BRAC...	STS3.5x3...	VBrace	Tube	A36 Gr.36	Typical	3.73	6.812	6.09	10.12
6	BF-BRAC...	STS6x6x...	VBrace	Tube	A36 Gr.36	Typical	5.59	31.745	30.3	47.527

Node Loads and Enforced Displacements (BLC 3 : E)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (... Inactive [(k, k-ft), (in,...
1	N2	L	X	16.92 Active
2	N5	L	X	45 Active
3	N7	L	X	22.3 Active

Story Drift X-Service

No Data to Print...

Story Drift Z-Service

No Data to Print...

Story Drift X-Strength

	Story (El...		Story Dri...	Loc	LC	Drift Rati...	Loc	LC	2nd/1st...	Loc	LC
1	42 ft	max	0.2832	0, 29.3	3	0.1644	0, 29.3	6	1.0004	0, 29.3	5
2		min	-0.2861	0, 29.3	6	0.0001	0, 29.3	1	1	0, 29.3	1
3	27.5 ft	max	0.4883	0, 0	3	0.3031	0, 0	6	1.0007	0, 0	6
4		min	-0.4911	0, 0	6	0.0001	0, 0	1	1	0, 0	1
5	14 ft	max	0.5184	0, 0	3	0.3092	0, 0	6	1.0007	0, 0	6
6		min	-0.5194	0, 0	6	0	0, 0	1	1	0, 0	1
7	0 ft	max	0	0, 0	6	NC			1	0, 0	6
8		min	0	0, 0	1	NC			1	0, 0	1

Story Drift Z-Strength

	Story (El...		Story Dri...	Loc	LC	Drift Rati...	Loc	LC	2nd/1st...	Loc	LC
1	42 ft	max	NC			NC			NC		
2		min	NC			NC			NC		
3	27.5 ft	max	NC			NC			NC		
4		min	NC			NC			NC		
5	14 ft	max	NC			NC			NC		
6		min	NC			NC			NC		
7	0 ft	max	NC			NC			NC		
8		min	NC			NC			NC		

Member Distributed Loads (BLC 1 : DL)

	Member Label	Direction	Start Magnitud...	End Magnitude...	Start Location [...	End Location [(...	Inactive [(k, k-f...
1	M3	Y	-0.2	-0.2	0	%100	Active
2	M4	Y	-0.9	-0.9	0	%100	Active
3	M5	Y	-0.9	-0.9	0	%100	Active

Member Distributed Loads (BLC 2 : LL)

	Member Label	Direction	Start Magnitud...	End Magnitude...	Start Location [...	End Location [(...	Inactive [(k, k-f...
1	M5	Y	-0.5	-0.5	0	%100	Active
2	M4	Y	-0.5	-0.5	0	%100	Active
3	M3	Y	-0.2	-0.2	0	%100	Active

Maximum Member Section Forces

	Me...		Axi...	Loc...	LC	y S...	Loc...	LC	z S...	Loc...	LC	Tor...	Loc...	LC	y-y...	Loc...	LC	z-z...	Loc...	LC
1	M1	max	79....	0	6	0.4...	27....	3	0	42	6	0	42	6	0	42	6	5.5...	27....	6
2		min	-17....	27....	3	-0.4...	14	6	0	0	1	0	0	1	0	0	1	-5.4...	27....	3
3	M2	max	77....	0	5	0.3...	27....	5	0	42	6	0	42	6	0	42	6	4.1...	27....	4
4		min	-14....	27....	4	-0.3...	14	4	0	0	1	0	0	1	0	0	1	-4.2...	27....	5
5	M3	max	17....	14.65	3	3.9...	14....	5	0	29.3	6	0	29.3	6	0	29.3	6	10....	14.65	5
6		min	-17....	0	6	-3.9...	14.65	5	0	0	1	0	0	1	0	0	1	-8.4...	6.1...	6
7	M4	max	20....	14.65	3	21....	0	5	0	29.3	6	0	29.3	6	0	29.3	6	0	29.3	6
8		min	-22....	0	6	-21....	29.3	5	0	0	1	0	0	1	0	0	1	-15...	14.65	5
9	M5	max	22....	14.65	3	12....	14....	6	0	29.3	6	0	29.3	6	0	29.3	6	34....	14.65	6
10		min	-22....	0	6	-13....	14.65	6	0	0	1	0	0	1	0	0	1	-24....	5.799	5
11	M6	max	18....	0	6	0.0...	0	4	0	20....	6	0	20....	6	0	20....	6	0	20....	6
12		min	-9.118	20....	3	-0.0...	20....	5	0	0	1	0	0	1	0	0	1	-0.484	10....	4
13	M7	max	18....	20....	5	0.0...	0	3	0	20....	6	0	20....	6	0	20....	6	0	20....	6
14		min	-9.2...	0	4	-0.0...	20....	4	0	0	1	0	0	1	0	0	1	-0.4...	10....	3
15	M10	max	72....	0	6	0.1...	0	6	0	20....	6	0	20....	6	0	20....	6	0	20....	6
16		min	-46....	20....	3	-0.1...	20....	4	0	0	1	0	0	1	0	0	1	-0.7...	10....	3
17	M11	max	75....	20....	5	0.1...	0	6	0	20....	6	0	20....	6	0	20....	6	0	20....	6
18		min	-49....	0	4	-0.1...	20....	3	0	0	1	0	0	1	0	0	1	-0.6...	10....	6
19	M12	max	40....	19....	3	0.11...	0	2	0	19....	6	0	19....	6	0	19....	6	0	19....	6
20		min	-47....	0	6	-0.1...	19....	5	0	0	1	0	0	1	0	0	1	-0.5...	9.9...	2
21	M13	max	37....	0	4	0.11...	0	4	0	19....	6	0	19....	6	0	19....	6	0	19....	6
22		min	-43....	19....	5	-0.1...	19....	6	0	0	1	0	0	1	0	0	1	-0.5...	9.9...	4

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AISC Eighth Edition Column Design Wide Flange Member Only		Project ID:	19051
			Project Name:	ATC
			Engineer:	HL
			Date:	11/20/2020

Member # = **BF1-B2**

Shape **W 18x55**

Properties	Geometry	
	A	16.2 <i>in.²</i>
	d	18.11 <i>in.</i>
	b_f	7.53 <i>in.</i>
	t_f	0.63 <i>in.</i>
	t_w	0.39 <i>in.</i>
	W(weight)	55 <i>#/ft.</i>
	r_T	1.95
	d/A_T	3.82

Compact Section Criteria	
b_f/2t_f	6
F_y	— <i>ksi</i>
d/t_w	46.4
F_y	— <i>ksi</i>
F_y	30.6 <i>ksi</i>

Elastic Properties					
X-X			Y-Y		
I_x	890	<i>in.⁴</i>	I_y	44.9	<i>in.⁴</i>
S_x	98.3	<i>in.³</i>	S_y	11.9	<i>in.³</i>
r_x	7.41	<i>in.</i>	r_y	1.67	<i>in.</i>
Plastic Modulus					
Z_x	112	<i>in.³</i>	Z_y	18.5	<i>in.³</i>

Torsional Constant	
J	1.66 <i>in.⁴</i>

Warping Constant	
C_w	— <i>in.⁶</i>

Material Property	
E	29000 <i>ksi</i>
F_y	36 <i>ksi</i>

Non-Composite Beam/Column Design AISC Eighth Edition

Building info:

Steel Properties

Total Member Length =	29.33	ft.
Beam Spacing (w) =	0	ft.
Tributary Area to Col=	0	ft.^2
K=	1	
Unbraced Length =	14.5	ft.

Loading

Dead Load (D)=	90	psf
Live Load(L)=	50	psf
Roof Live Load(Lr)=	0	psf
Snow Load(S)=	25	psf

Excentricity on Column

ex =	2	in.
ey =	0	in.

Deflection Criteria

Total Load	L/360=	0.98	in.
------------	--------	------	-----

Max Moment About Major Axis:

MD(Max Dead)=	0.2	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0.2	kips-in
ML(Max Live)=	0.2	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0.2	kips-in
M(Lr or S)(Max Live)=	0.2	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0.2	kips-in
Mu=	1896	kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Shear

VD(Max Dead)=	0	kips
VD(Dead Overwrite)=	0	kips
VMD(Max Use)=	0	kips
VL(Max Live)=	0	kips
VL(Live Overwrite)=	0	kips
VL(Max Use)=	0	kips
V(Lr or S)(Max Live)=	0	kips
V(Lr or S)(Live Overwrite)=	0	kips
V(Lr or S)(Max Use)=	0	kips
Vu=	20	kips

Max Axial Load

PD(Max Dead)=	0	kips
PD(Dead Overwrite)=	0.1	kips
PD(Max Use)=	0.1	kips
PL(Max Live)=	0	kips
PL(Live Overwrite)=	0.1	kips
PL(Max Use)=	0.1	kips
P(Lr or S)(Max Live)=	0	kips
P(Lr or S)(Live Overwrite)=	0.1	kips
P(Lr or S)(Max Use)=	0.1	kips
P(E or W)=	23	kips
Pu=	23	kips

Solution Summary:

Try **W 18x55** <---- Change W shapes

Member	Major Axis Beam Bending	56%
	Beam Shear	19%
	Minor Axis Beam Bending	0%
	Axial	41%
	Deflection	0%
	Combined Bending and Axial	98%
Overall Check		98%

Column Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Column Height= 29.33 ft
K= 1
KL= 29.33 ft
KL/r= 210.8

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	126
if $\frac{KL}{r} \leq C_c$ then $F_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}]F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 15.1.3.1
if $\frac{KL}{r} \geq C_c$ then $F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 15.1.3.2
Column in Tension	
$F_a = 0.6F_y$	Section 15.1.1

Fa = 3.4 ksi

Calculate Stresses in Column

Axial Stress: fa= Pu/A= 1.4 ksi Okay Increases by 1/3

Non-Composite Major Axis Beam Design AISC Eighth Edition

Unbraced length (Lb)= 14.5 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f/2t_f \leq 65/\sqrt{F_y}$	6	10.8	Good
$d/t_w \leq 412(1 - 2.33\frac{f_a}{F_y})/\sqrt{F_y}$	46.4	62.4	Good
$l_b \leq 76b_f/\sqrt{F_y}$	174	95.4	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d/A_f)F_y}$	174	145.5	No Good, Use Alternate EQN
			Use Alternate Equation

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2/\sqrt{F_y} < b_f/2t_f$	6	8.7	Good
$b_f/2t_f \leq 95.0/\sqrt{F_y}$	6	15.8	Good
			Use Alternate Bending Stress Equation

Typical Bending Stress Equation: $F_{bx} = 0.66F_y$

Fbx= N/A ksi Increases by 1/3

Alternate Bending Stress Equation: $F_{bx} = F_y[0.733 - 0.0014(\frac{b_f}{t_f})\sqrt{F_y}]$

Fbx= 31.92 ksi Increases by 1/3

Shear Stress Equation: $F_v = 0.40F_y$

Fv= 14.4 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.75F_y$

Fby= 27 ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	17.9	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w*d) =$	2.8	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5wL^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

$C_{mx} =$	1
$C_{my} =$	1

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{C_{mx}f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{C_{my}f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.98	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.63	Less than 1.0, Okay

AISC Eighth Edition Column Design Wide Flange Member Only

Project ID:	19051
Project Name:	ATC
Engineer:	HL
Date:	11/20/2020

Member # = **BF1-C1**

Shape **W 14x61**

Properties	Geometry	
	A	17.9 <i>in.²</i>
	d	13.89 <i>in.</i>
	b_f	9.995 <i>in.</i>
	t_f	0.645 <i>in.</i>
	t_w	0.375 <i>in.</i>
	W(weight)	61 <i>#/ft.</i>
	r_T	2.7
	d/A_f	2.15

Compact Section Criteria	
b_f/2t_f	7.7
F_y	— <i>ksi</i>
d/t_w	37
F_y	— <i>ksi</i>
F_y	48.1 <i>ksi</i>

Elastic Properties					
X-X			Y-Y		
I_x	640	<i>in.⁴</i>	I_y	107	<i>in.⁴</i>
S_x	92.2	<i>in.³</i>	S_y	21.5	<i>in.³</i>
r_x	5.98	<i>in.</i>	r_y	2.45	<i>in.</i>
Plastic Modulus					
Z_x	102	<i>in.³</i>	Z_y	32.8	<i>in.³</i>

Torsional Constant	
J	2.2 <i>in.⁴</i>

Warping Constant	
C_w	— <i>in.⁶</i>

Material Property	
E	29000 <i>ksi</i>
F_y	36 <i>ksi</i>

Non-Composite Beam/Column Design AISC Eighth Edition

Building info:

Steel Properties

Total Member Length =	14.5	ft.
Beam Spacing (w) =	0	ft.
Tributary Area to Col=	0	ft.^2
K=	1	
Unbraced Length =	14.5	ft.

Loading

Dead Load (D)=	80	psf
Live Load(L)=	60	psf
Roof Live Load(Lr)=	0	psf
Snow Load(S)=	25	psf

Excentricity on Column

ex =	2	in.
ey =	0	in.

Deflection Criteria

Total Load	L/360=	0.48	in.
------------	--------	------	-----

Max Moment About Major Axis:

MD(Max Dead)=	0.2	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0.2	kips-in
ML(Max Live)=	0.2	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0.2	kips-in
M(Lr or S)(Max Live)=	0.2	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0.2	kips-in
Mu=	72	kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Shear

VD(Max Dead)=	0	kips
VD(Dead Overwrite)=	0	kips
VMD(Max Use)=	0	kips
VL(Max Live)=	0	kips
VL(Live Overwrite)=	0	kips
VL(Max Use)=	0	kips
V(Lr or S)(Max Live)=	0	kips
V(Lr or S)(Live Overwrite)=	0	kips
V(Lr or S)(Max Use)=	0	kips
Vu=	1	kips

Max Axial Load

PD(Max Dead)=	0	kips
PD(Dead Overwrite)=	0.1	kips
PD(Max Use)=	0.1	kips
PL(Max Live)=	0	kips
PL(Live Overwrite)=	0.1	kips
PL(Max Use)=	0.1	kips
P(Lr or S)(Max Live)=	0	kips
P(Lr or S)(Live Overwrite)=	0.1	kips
P(Lr or S)(Max Use)=	0.1	kips
P(E or W)=	80	kips
Pu=	80	kips

Solution Summary:

Try **W 14x61** <---- Change W shapes

Member	Major Axis Beam Bending	3%
	Beam Shear	1%
	Minor Axis Beam Bending	0%
	Axial	28%
	Deflection	0%
	Combined Bending and Axial	30%
Overall Check		30%

Column Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Column Height= 14.5 ft
K= 1
KL= 14.5 ft
KL/r= 71

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	126
if $\frac{KL}{r} \leq C_c$ then $F_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
if $\frac{KL}{r} \geq C_c$ then $F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2
Column in Tension	
$F_a = 0.6F_y$	Section 1.5.1.1

Fa = 16.3 ksi

Calculate Stresses in Column

Axial Stress: fa= Pu/A= 4.5 ksi Okay Increases by 1/3

Non-Composite Major Axis Beam Design AISC Eighth Edition

Unbraced length (Lb)= 14.5 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 65 / \sqrt{F_y}$	7.7	10.8	Good
$d / t_w \leq 412(1 - 2.33 \frac{f_u}{F_y}) / \sqrt{F_y}$	37	48.7	Good
$l_b / \leq 76 b_f / \sqrt{F_y}$	174	126.6	No Good, Use Alternate EQN
$l_b / \leq \frac{20,000}{(d/A_f) F_y}$	174	257.9	Good
			Use Alternate Equation

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	7.7	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	7.7	15.8	Good
			Use Alternate Bending Stress Equation

Typical Bending Stress Equation: $F_{bx} = 0.66F_y$

Fbx= N/A ksi Increases by 1/3

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014(\frac{b_f}{t_f}) \sqrt{F_y}]$

Fbx= 31.92 ksi Increases by 1/3

Shear Stress Equation: $F_v = 0.40F_y$

Fv= 14.4 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.75F_y$

Fby= 27 ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	0.8	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w*d) =$	0.2	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

C _{mx} =	1
C _{my} =	1

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{C_{mx}f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{C_{my}f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.3	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.23	Less than 1.0, Okay

SAMPLE BRACE DESIGN

AISC Eighth Edition HSS Brace Member Only

Project ID:	19051
Project Name:	ATC
Engineer:	HL
Date:	2/20/2021

Member # = BF- Brace

Shape ST S 6x6x0.25

Properties	Geometry	
	A	5.59 in. ²
	d	6 in.
	b	6 in.
	t	0.25 in.
	W(weight)	19.02 #/ft.

Compact Section Criteria	
b/t	24
h/t	24

Material Property	
E	29000 ksi
Fy	36 ksi

Elastic Properties					
X-X			Y-Y		
I _x	30.3	in. ⁴	I _y	30.3	in. ⁴
S _x	10.1	in. ³	S _y	10.1	in. ³
r _x	2.33	in.	r _y	2.33	in.

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v3B>

Non-Composite Brace Design AISC Eighth Edition

Building info:

Steel Properties

Brace Length=	20.83	ft
Tributary Area to Col=	0	ft^2
K factor =	1	

Loading

Dead Load (D)=	90	psf
Live Load(L)=	50	psf
Roof Live Load(Lr)=	0	psf
Snow Load(S)=	25	psf

Excentricity on Column

ex =	0	in
ey =	0	in

Max Moment About Major Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Axial Load

PD(Max Dead)=	0	kips
PD(Dead Overwrite)=	0	kips
PD(Max Use)=	0	kips
PL(Max Live)=	0	kips
PL(Live Overwrite)=	0	kips
PL(Max Use)=	0	kips
P(Lr or S)(Max Live)=	0	kips
P(Lr or S)(Live Overwrite)=	0	kips
P(Lr or S)(Max Use)=	0	kips
P(E or W)=	76	kips
Pu=	76	kips

Solution Summary:Try **ST S 6x6x0.25** Change Tube shapes

Compactness

OK b/t 31.67
OK d/t 42.83

Member	Major Axis Brace Bending	0%
	Minor Axis Brace Bending	0%
	Axial	85%
	Combined Bending and Axial	85%
Overall Check		85%

Brace Design AISC Eighth Edition

Brace Length= 20.83 ft
 K= 1
 KL= 20.83 ft
 KL/r= 107.3

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	126
$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2
Column in Tension	
$F_a = 0.6F_y$	section 1.5.1.1

Fa = 15.96 ksi Note if Seismic/Wind loads occurs, 1/3 allowable stress will be used

Calculate Stresses in Braces

Axial Stress: fa= Pu/A 13.6 ksi Column Okay

Brace Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Checks for Typical Major Bending Stress Equation		
Equation	Variable Value	Limit Value
$b/t \leq 238 / \sqrt{F_y}$	24	39.7
Major Axis Bending Equation: $F_{bx} = 0.6F_y$		Fbx= 28.728 ksi

Non-Composite Minor Axis Beam/Brace Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.6F_y$ Fby = 28.728 ksi

Calculate Stresses in Beam/Brace

Major Bending Stress: fbx= Mux/Sx=	0	ksi	Column Okay
Minor Bending Stress: fby= Muy/Sy=	0	ksi	Column Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.85	Less than 1.0, Column Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.63	Less than 1.0, Column Okay

AISC Eighth Edition ORDINARY CONCENTRIC
BRACED FRAME CONNECTION DESIGN

Project ID:	19051
Project Name:	ATC
Engineer:	HL
Date:	2/20/2021

Braced Frame # 2nd Floor

Brace Size: ST S 6x6x0.25

Brace Properties	
A	5.59 in ²
t	0.25 in
B	6 in
F _y	36 ksi
Ultimate Axial Load applied to Brace=	77.1 kips

Weld Properties	
D	4/16"

Gusset Plate Properties	
tp	0.625 in
F _{yp}	36 ksi
E	29000 ksi
K	0.7
L _p	8 in
L	8 in
r	0.18 in
KL/r	31.1
C _c	126

Brace Yielding	$P_a = 0.6F_y A_{net} =$	114	kips	Okay
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Required Weld Length	$L_w = \frac{P_a}{0.928D * 4} =$	5	in
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Gusset Plate Yielding	$P_a = 0.6F_{yp} A_p =$	108	kips	Okay
-----------------------	-------------------------	-----	------	------

Gusset Plate Buckling	$P_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}]F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - \frac{(\frac{KL}{r})^3}{8C_c^3}} A_p =$	99.3	kips	Okay
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Design Case D

2018 IBC Risk Category III Seismic Design Category C

This section presents representative structural calculations for Building 3 (3-story education facility in St. Louis, Missouri) Design Case D. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v3A.

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DESIGN CRITERIA

Search Information

Address: St. Louis, MO, USA

Coordinates: 38.62700249999999, -90.19940419999999

Elevation: 462 ft

Timestamp: 2020-10-07T21:45:59.452Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: III

Site Class: D

MCE_R Horizontal Response Spectrum

Design Horizontal Response Spectrum

Basic Parameters

Name	Value	Description
S _S	0.453	MCE _R ground motion (period=0.2s)
S ₁	0.16	MCE _R ground motion (period=1.0s)
S _{MS}	0.651	Site-modified spectral acceleration value
S _{M1}	0.365	Site-modified spectral acceleration value
S _{DS}	0.434	Numeric seismic design value at 0.2s SA
S _{D1}	0.244	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F _a	1.438	Site amplification factor at 0.2s
F _v	2.279	Site amplification factor at 1.0s
CR _S	0.909	Coefficient of risk (0.2s)
CR ₁	0.863	Coefficient of risk (1.0s)
PGA _G	0.261	MCE _G peak ground acceleration
F _{PGA}	1.339	Site amplification factor at PGA
PGA _M	0.349	Site modified peak ground acceleration
T _L	12	Long-period transition period (s)
SsRT	0.453	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.498	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.16	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.186	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGA _d	0.5	Factored deterministic acceleration value (PGA)

Search Information

Address: St. Louis, MO, USA
Coordinates: 38.62700249999999, -90.19940419999999
Elevation: 462 ft
Timestamp: 2020-08-24T21:13:20.719Z
Hazard Type: Wind

ASCE 7-16

MRI 10-Year 74 mph
MRI 25-Year 81 mph
MRI 50-Year 86 mph
MRI 100-Year 93 mph
Risk Category I 101 mph
Risk Category II 106 mph
Risk Category III 114 mph
Risk Category IV 118 mph

ASCE 7-10

MRI 10-Year 76 mph
MRI 25-Year 84 mph
MRI 50-Year 90 mph
MRI 100-Year 96 mph
Risk Category I 105 mph
Risk Category II 115 mph
Risk Category III-IV 120 mph

ASCE 7-05

ASCE 7-05 Wind Speed 90 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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Search Information

Address: St. Louis, MO, USA
Coordinates: 38.62700249999999, -90.19940419999999
Elevation: 462 ft
Timestamp: 2020-08-24T21:23:22.173Z
Hazard Type: Snow

ASCE 7-16

ASCE 7-10

ASCE 7-05

Ground Snow Load 20 lb/sqft Ground Snow Load 20 lb/sqft Ground Snow Load 20 lb/sqft

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer.

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DESIGN CRITERIA

VERTICAL LOADS

AREA	DESIGN DEAD LOAD	LIVE LOAD (1)	CONCENTRATED LOADS
CLASSROOM AND OFFICE	75 PSF	50 PSF	1000#
CORRIDORS ABOVE 1ST FLOOR	75 PSF	80 PSF	1000#
ROOF	20 PSF	20 PSF	-

(1) LIVE LOADS EXCEPT SNOW LOADS ARE REDUCED PER IBC SECTION 1607.11.

SNOW:

$P_g = 20$ PSF = GROUND SNOW LOAD

$P_f = 0.7 C_e C_t I_s P_g$ = FLAT ROOF SNOW LOAD

$I_s = 1.1$ $C_e = 1.0$, $C_t = 1.0$. C_s = VARIES

VIBRATION DESIGN CRITERIA:

VIBRATION CONSIDERATIONS WERE DESIGNED PER THE RECOMMENDATIONS IN THE GUIDELINES FOR DESIGN AND CONSTRUCTION OF COMPUTER EQUIPMENT RESIDENCES. FOR CLASSROOM FLOORS A VIBRATION VELOCITY LIMIT OF 8000 MICRO INCHES PER SECOND WAS EVALUATED AT A WALKING SPEED OF 75 STEPS PER MINUTE.

LATERAL FORCES

LATERAL FORCES ARE TRANSMITTED BY DIAPHRAGM ACTION OF ROOF AND FLOORS TO BRACED FRAME/MOMENT FRAME. LOADS ARE THEN TRANSFERRED TO FOUNDATION BY BRACED FRAME/MOMENT FRAME ACTION WHERE ULTIMATE DISPLACEMENT IS RESISTED BY PASSIVE PRESSURE OF EARTH AND/OR SLIDING FRICTION. OVERTURNING IS RESISTED BY DEAD LOAD OF THE STRUCTURE.

WIND:

THE BUILDING MEETS THE CRITERIA TO USE THE "ENCLOSED, PARTIALLY ENCLOSED, AND OPEN BUILDING OF ALL HEIGHTS PROCEDURE" PER ASCE 7-16.

- EXPOSURE CATEGORY = B
- BASIC WIND SPEED, (3 SEC. GUST), $V_{ULT} = 114$ MPH; $V_{ASD} = 88$ MPH
- RISK CATEGORY PER IBC TABLE 1604.5 = III
- TOPOGRAPHIC FACTOR $K_{ZT} = 1.0$
- INTERNAL PRESSURE COEFFICIENT (ENCLOSED) = ± 0.18
- BASE SHEAR $V = 146$ KIPS AT BRACE FRAMES, 64 KIPS AT MOMENT FRAME

SEISMIC: (ASCE 7-16) $V = C_s W$

WHERE $C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$; WITH

C_s MINIMUM = $0.044 S_{DS} I_e \geq 0.01$

OR

C_s MINIMUM = $\frac{0.5 S_1}{\frac{R}{I_e}}$ FOR $S_1 > 0.6g$

C_s MAXIMUM = $\frac{S_{D1}}{T \left(\frac{R}{I_e}\right)}$ FOR $T \leq T_L$

OR

C_s MAXIMUM = $\frac{S_{D1} T_L}{T^2 \left(\frac{R}{I_e}\right)}$ FOR $T > T_L$

SEISMIC IMPORTANCE FACTOR, $I_e = 1.25$

RISK CATEGORY OF BUILDING PER IBC TABLE 1604.5 = III

SPECTRAL RESPONSE ACCELERATIONS $S_s = 0.453$ & $S_1 = 0.16$

SITE CLASS PER TABLE 20.3-1 = D

DESIGN SPECTRAL RESPONSE ACCELERATIONS $S_{DS} = 0.434$ & $S_{D1} = 0.243$

SEISMIC DESIGN CATEGORY = C

W = EFFECTIVE SEISMIC WEIGHT OF BUILDING = 2140 KIPS

ANALYSIS PROCEDURE USED = EQUIVALENT LATERAL FORCE PROCEDURE/MODAL RESPONSE SPECTRUM

ANALYSIS

RESPONSE MODIFICATION FACTOR PER TABLE 12.2-1, $R = 8.0$ AT SPECIAL STEEL MOMENT FRAMES, $R = 6.0$ AT

SPECIAL CONCENTRICALLY STEEL BRACED FRAMES

$C_s = 0.047$ AT MOMENT FRAMES, $C_s = 0.09$ AT BRACED FRAMES

DESIGN BASE SHEAR $V = 188.4$ KIPS AT BRACED FRAMES, 97.62 KIPS AT MOMENT FRAMES

FOUNDATION DESIGN CRITERIA

SOIL BEARING PRESSURE: 4000 PSF (ASSUMED)*

ACTIVE PRESSURE - RESTRAINED: 50 PCF +14H SEISMIC SURCHARGE (ASSUMED)

ACTIVE PRESSURE - UNRESTRAINED: 35 PCF +6H SEISMIC SURCHARGE (ASSUMED)

PASSIVE RESISTANCE: 200 PCF (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

COEFFICIENT OF FRICTION: .35 (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

*1/3 INCREASE ALLOWED FOR SEISMIC OR WIND LOADING

ITEM	DESIGN f'_c (PSI) (AT 28 DAYS U.N.O.)
SLABS ON GRADE - UNO	4000
FOUNDATIONS - UNO	3000
STEM WALLS AND OTHER WALLS - UNO	4000
ELEVATED DECKS, TOPPING SLABS, AND SLABS ON METAL	5000

GRAVITY DESIGN

NOTE: BENTLEY RAM STRUCTURAL WAS USED AS DESIGN AIDS

ROOF BEAM DESIGN

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Floor Map

02/18/21 12:47:46

Steel Code: AISC 360-10 LRFD

Floor Type: Roof

Beam Numbers

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Beam Design

02/18/21 12:47:46

Steel Code: AISC 360-10 LRFD

Floor Type: Roof**Beam Number = 138****SPAN INFORMATION (ft): I-End (68.75,2.67) J-End (68.75,32.00)**

Beam Size (Optimum) = W16X26

Fy = 50.0 ksi

Total Beam Length (ft) = 29.33

Mp (kip-ft) = 184.17

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.190	0.000	---	Snow	0.000
	29.333	0.190	0.000			0.000
2	0.000	0.026	0.000	---	NonR	0.000
	29.333	0.026	0.000			0.000

SHEAR (Ultimate): Max Vu (1.4DL) = 4.44 kips 0.90Vn = 105.97 kips**MOMENTS (Ultimate):**

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.4DL	32.5	14.7	0.0	1.00	0.90	165.75
Controlling		1.4DL	32.5	14.7	0.0	1.00	0.90	165.75

REACTIONS (kips):

	Left	Right
DL reaction	3.17	3.17
Max +total reaction (factored)	4.44	4.44

DEFLECTIONS:

					Ratio
Dead load (in)	at 14.67 ft =	-0.412	L/D = 853 > 240		0.28
Live load (in)	at 14.67 ft =	-0.000			
Net Total load (in)	at 14.67 ft =	-0.412	L/D = 853 > 240		0.28

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FLOOR BEAM DESIGN

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Floor Map

11/18/20 09:27:36

Steel Code: AISC 360-10 LRFD

Floor Type: LEVEL 2

Beam Numbers

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Beam Design

02/11/21 15:54:03

Steel Code: AISC 360-10 LRFD

Floor Type: LEVEL 2**Beam Number = 135****SPAN INFORMATION (ft): I-End (59.25,2.67) J-End (59.25,32.00)**

Beam Size (Optimum) = W16X26

Fy = 50.0 ksi

Total Beam Length (ft) = 29.33

COMPOSITE PROPERTIES (Not Shored):

		Left	Right
Deck Label		3W+2.5	3W+2.5
Concrete thickness (in)		2.50	2.50
Unit weight concrete (pcf)		150.00	150.00
f _c (ksi)		4.00	4.00
Decking Orientation		perpendicular	perpendicular
Decking type		ASC 3W	ASC 3W
beff (in)	=	88.00	Y bar(in) = 17.42
M _{nf} (kip-ft)	=	406.66	M _n (kip-ft) = 280.02
C (kips)	=	103.38	PNA (in) = 11.99
I _{eff} (in ⁴)	=	770.41	I _{tr} (in ⁴) = 1205.71
Stud length (in)	=	4.50	Stud diam (in) = 0.75
Stud Capacity (kips)	Q _n = 17.2	R _g = 1.00	R _p = 0.60
# of studs:	Full = 52	Partial = 12	Actual = 12
Number of Stud Rows = 1	Percent of Full Composite Action = 26.92		

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.731	0.487	0.487	12.3%	Red	0.146	0.195
	29.333	0.731	0.487	0.487			0.146	0.195
2	0.000	0.026	0.026	0.000	---	NonR	0.000	0.000
	29.333	0.026	0.026	0.000			0.000	0.000

SHEAR (Ultimate): Max V_u (1.2DL+1.6LL) = 26.80 kips 0.90V_n = 105.97 kips**MOMENTS (Ultimate):**

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	99.8	14.7	0.0	1.00	0.90	165.75
	Init DL	1.4DL	77.3	14.7	---	---		
	Max +	1.2DL+1.6LL	196.5	14.7	---	---	0.90	252.02
Controlling		1.2DL+1.6LL	196.5	14.7	---	---	0.90	252.02

REACTIONS (kips):

	Left	Right
Initial reaction	10.39	10.39
DL reaction	11.11	11.11
Max +LL reaction	8.42	8.42
Max +total reaction (factored)	26.80	26.80

DEFLECTIONS: (Camber = 3/4)

			Ratio
Initial load (in)	at 14.67 ft =	-0.980	L/D = 359 > 200 0.56
Live load (in)	at 14.67 ft =	-0.428	L/D = 823 > 360 0.44

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RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Beam Design

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Steel Code: AISC 360-10 LRFD

Post Comp load (in)	at 14.67 ft = -0.610	L/D = 577 > 240	0.42
Net Total load (in)	at 14.67 ft = -0.840	L/D = 419 > 240	0.57

VIBRATION TO A TYPICAL BAY

FloorVibe V3.00, (C)2006-2018 by Structural Engineers, Inc.
www.floorvibe.com
Licensee: PCS Structural Solutions, Tacoma, Washington

SHEET OF
DATE: 6/25/2020
PAGE: 1 BY: HL

Project ID: ATC-NIST Bldg Comparison
Project # : 19051
Bay ID : Typ Bay

VIBRATION ANALYSIS:

Activity: Walking
Occupancy Category: Electronic Office
Evaluation Criterion: Walking
References: Murray, T.M., Allen, D.E., Ungar, E.E, Davis, D.B.
"Vibrations of Steel-Framed Structural Systems
Due To Human Activity", AISC Design Guide #11 2nd Ed, 2016
Murray, T.M., and Davis, B.,
"Vibration of Steel Joist-Concrete Floor Systems",
SJI Technical Digest No. 5, 2014
Constant Force, $P_o = 65.1b$
Modal Damping Ratio, $\beta = 0.025$
Acceleration Limit, $a_o/g \times 100\% = 0.50 \%$
Beams are not continuous
Girders are not continuous at columns

EVALUATION: Combined mode $a_p/g = 0.514 \%$ > 0.50%
This peak acceleration MAY BE ACCEPTABLE,
ENGINEERING JUDGEMENT IS REQUIRED.

PARAMETER SUMMARY:

Parameter	Beam Panel	Left Girder Panel	Right Girder Panel	System
Member	W16X40	W18X50	W18X50	
Uniform Load, plf	663.3	1307.0	1003.5	
Effective I, in ⁴	1924.0	2790.3	2507.2	
Frequency, Hz	8.27	6.52	7.05	5.12
Panel Width, ft	26.75	50.67	50.67	
Panel Weight, k	51.0	104.8	106.1	84.2

FRAMING:

Girder Span = 30.00 ft
Beam Spans:
Left = 9.15 ft
Center = 28.75 ft
Right = 0.00 ft
Girders/Walls:
Left -W18X50
Right-W18X50
Beam -W16X40
3 spaces at 120.00 in
Floor Width =120.00 ft
Floor Length= 76.00 ft
Concrete: dc = 5.50 in
f'c = 3.00 ksi
wt. = 145.0 pcf
Deck Height = 3.00 in

Loading: Dead = 4.00 psf
Live = 8.00 psf
Collateral = 0.00 psf

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This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v3B>

Note: Floors with a frequency below 3 Hz are not recommended

COLUMN DESIGN

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Floor Map

11/18/20 09:27:36

Steel Code: AISC 360-10 LRFD

Floor Type: LEVEL 2

Beam Numbers

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Column Design

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Steel Code: AISC 360-10 LRFD

Story level Level 2, Column Line 6-C, Column # 31

Fy (ksi) = 50.00

Column Size

= W10X33

Orientation (deg.) = 90.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	13.75	13.75
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	7.36	6.48
Bottom _____	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	72.01	31.85	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 137.37	0.90Pnx (kip) = 390.25	Pu/0.90Pnx = 0.352
	0.90Pny (kip) = 257.68	Pu/0.90Pny = 0.533
	0.90Pn (kip) = 257.68	Pu/0.90Pn = 0.533

DEMAND/CAPACITY LIMIT FOR STRENGTH : 1.000**CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 10:**

	Dead	Live	Roof
Axial (kip) _____	72.01	28.24	0.00
Moments Top Mx (kip-ft) _____	-2.32	-1.20	0.00
My (kip-ft) _____	-0.61	-1.28	0.00
Bot Mx (kip-ft) _____	0.00	0.00	0.00
My (kip-ft) _____	0.00	0.00	0.00

Single curvature about X-Axis

Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 131.59	0.90*Pn (kip) = 257.68
Mux (kip-ft) = -4.71	0.90*Mnx (kip-ft) = 145.50
Muy (kip-ft) = -2.78	0.90*Mny (kip-ft) = 52.50
Rm = 1.00	
Cbx = 1.67	
Cmx = 0.60	Cmy = 0.60
Pex (kip) = 1797.74	Pey (kip) = 384.78
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.511

RAM Structural System

Gravity Column Design

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

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Steel Code: AISC 360-10 LRFD

$$\text{Eq H1-1a: } 0.511 + 0.029 + 0.047 = 0.587$$

LATERAL DESIGN

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SEISMIC FORCE

IBC 2018 RISK Category III Education Building

From Spreadsheet,

N-S Direction, SCBF used as force resist element, R=6, Omega=2, Cs=0.09

E-W Direction, SMF used as force resist element, R=8, Omega=3, Cs=CsMax=0.067

N-S:

$$I_{ns} := 1.25$$

$$R_{ns} := 6$$

From Table 12.2-1

$$\Omega_{ns} := 2.0$$

$$Cd_{ns} := 5$$

$$h_1 := 13.75$$

$$\text{Height1} := 13.75 \quad \text{ft}$$

$$h_2 := 27.5 - 13.75 = 13.75$$

$$\text{Height2} := 13.75 \quad \text{ft}$$

$$h_3 := 14.5$$

$$\text{Height3} := 14.5 \quad \text{ft}$$

Allowable Story Drift: Δa_{ns}

$$\Delta a_{ns} := 0.020 \cdot h_{sx}$$

from Table 12.12-1

Max deflection(Drift):

from 12.8-15

Roof:

$$h_{sx} := h_1 \cdot 12$$

$$\Delta a_{ns} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ns} = 3.3$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ns}}{Cd_{ns}} = 0.825 \quad \text{in}$$

Third Floor:

$$h_{sx} := h_2 \cdot 12$$

$$\Delta a_{ns} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ns} = 3.3$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ns}}{Cd_{ns}} = 0.825 \quad \text{in}$$

Second Floor:

$$h_{sx} := h_3 \cdot 12$$

$$\Delta a_{ns} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ns} = 3.48$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ns}}{Cd_{ns}} = 0.87 \quad \text{in}$$

E-W:

From Table 12.2-1

$$I_{ew} := 1.25$$

$$R_{ew} := 8$$

$$\Omega_{ew} := 3$$

$$Cd_{ew} := 5.5$$

From Table 12.2-1

$$h_1 := 13.75 \quad \text{ft}$$

$$h_2 := 27.5 - 13.75 = 13.75 \quad \text{ft}$$

$$h_3 := 14.5 \quad \text{ft}$$

Allowable Story Drift: Δa_{ns}

$$\Delta a_{ew} := 0.02 \cdot h_{sx}$$

from Table 12.12-1

Max deflection(Drift):

from 12.8-15

Roof:

$$h_{sx} := h_1 \cdot 12$$

$$\Delta a_{ew} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ew} = 3.3$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ew}}{Cd_{ew}} = 0.75 \quad \text{in}$$

Third Floor:

$$h_{sx} := h_2 \cdot 12$$

$$\Delta a_{ew} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ew} = 3.3$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ew}}{Cd_{ew}} = 0.75 \quad \text{in}$$

Second Floor:

$$h_{sx} := h_3 \cdot 12$$

$$\Delta a_{ew} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ew} = 3.48$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ew}}{Cd_{ew}} = 0.791 \quad \text{in}$$

Check Seismic Weight:

$$DL_w := 20 \text{ psf}$$

Diaphragm 1:

Floor Dead Load	$DL_1 := 80 \text{ psf}$	(Partional load = 10psf included as Seismi Weight)
Floor Area	$FloorA_1 := 9860 \text{ sf}$	
Building Perimeter	$P_1 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$	
Wal Height	$H_1 := \frac{(Height2 + Height1)}{2} = 13.75 \text{ ft}$	
	$W_1 := (DL_1 \cdot FloorA_1 + P_1 \cdot H_1 \cdot DL_w) = 9.267 \times 10^5 \text{ lb}$	

Diaphragm 2:

Floor Dead Load	$DL_2 := 80 \text{ psf}$	(Partional load = 10psf included as Seismi Weight)
Floor Area	$FloorA_2 := 10100 \text{ sf}$	
Building Perimeter	$P_2 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$	
Wal Height	$H_2 := \frac{(Height2 + Height3)}{2} = 14.125 \text{ ft}$	
	$W_2 := DL_2 \cdot FloorA_2 + P_2 \cdot H_2 \cdot DL_w = 9.497 \times 10^5 \text{ lb}$	

Diaphragm 3(ROOF):

Floor Dead Load	$DL_3 := 20 \text{ psf}$	
Floor Area	$FloorA_3 := 9560 \text{ sf}$	
Building Perimeter	$P_3 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$	
Wal Height	$H_3 := \frac{(Height3)}{2} = 7.25 \text{ ft}$	
	$W_3 := DL_3 \cdot FloorA_3 + P_3 \cdot H_3 \cdot DL_w = 2.639 \times 10^5 \text{ lb}$	
	$\underline{W} := W_1 + W_2 + W_3 = 2.14 \times 10^6 \text{ lb}$	

Check Torsional Irregularity

By inspection, we notice there will be a torsional irregularity of Type 1a ($\delta_{avg} > 1.2(\text{Max}\delta)$) in Y direction
See next page for Torsional Irregularity Calc

$$\frac{\delta_{max}}{\delta_{avg}} = 1.347 \quad A_x := \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 = 1.26$$

TORSIONAL
AMPLIFICATION FACTOR
PER ASCE-7-16 12.8.4.3

When $A_x=1.0$ 5% of dimension of structure will be applied on both direction

Now in Y direction:

$$0.05 \cdot A_x = 0.063$$

New accidental moment 0.063% will be apply to design in Y direction

THIS HAS BEEN INPUTTED
TO RAM STRUCTURAL MODEL

Redundancy Factor p , per 12.3.4.2 permitted to be taken as 1.0 for SDC C building

RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

02/18/21 14:41:59

Loads and Applied Forces

LOAD CASE: EQ

Seismic ASCE 7-16 Equivalent Lateral Force

Importance Factor: 1.25 TL: 12.00 s

Site Class D: Stiff Soil, Default

Ss: 0.453 g S1: 0.160 g

Fa: 1.438 Fv: 2.280 SDs: 0.434 g SD1: 0.243 g

Use Specified Seismic Design Category: C

Provisions for: Force

Ground Level: Base

Dir	Eccent	R	Ta Equation		Building Period-T				
X	+ And -	8.0	Std,Ct=0.028,x=0.80		Calculated				
Y	+ And -	6.0	Std,Ct=0.030,x=0.75		Calculated				
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
X	0.559	1.457	1.372	0.814	0.068	0.047	0.024	0.047	1.157
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
Y	0.496	1.457	0.375	0.375	0.090	0.135	0.024	0.090	1.000

Total Building Weight (kips) = 2090.71

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	19.96	0.00	70.05	38.26
Level 3	1	27.50	54.03	0.00	72.11	40.86
Level 2	1	13.75	23.64	0.00	66.05	41.87

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	19.956	0.000
Level 3	1	54.028	0.000
Level 2	1	23.636	0.000
		97.62	0.00

APPLIED STORY FORCES

Type: EQ_ASCE716_X+E_F

Level	Ht ft	Fx kips	Fy kips
-------	----------	------------	------------

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RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

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Loads and Applied Forces

Roof	42.17	19.96	0.00
Level 3	27.50	54.03	0.00
Level 2	13.75	23.64	0.00
	<hr/>	<hr/>	
	97.62	0.00	

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	19.96	0.00	70.05	31.08
Level 3	1	27.50	54.03	0.00	72.11	33.02
Level 2	1	13.75	23.64	0.00	66.05	34.03

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	19.956	0.000
Level 3	1	54.028	0.000
Level 2	1	23.636	0.000
		<hr/>	<hr/>
		97.62	0.00

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	19.96	0.00
Level 3	27.50	54.03	0.00
Level 2	13.75	23.64	0.00
		<hr/>	<hr/>
		97.62	0.00

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	0.00	35.63	80.09	34.67
Level 3	1	27.50	0.00	103.16	82.15	36.94
Level 2	1	13.75	0.00	50.31	76.09	37.95

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx	Sum Fy
-------	---------	--------	--------

RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

Loads and Applied Forces

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		kip	kip
Roof	1	0.000	35.631
Level 3	1	0.000	103.157
Level 2	1	0.000	50.315
		<hr/> 0.00	<hr/> 189.10

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Ht ft	Fx kip	Fy kip
Roof	42.17	0.00	35.63
Level 3	27.50	0.00	103.16
Level 2	13.75	0.00	50.31
		<hr/> 0.00	<hr/> 189.10

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Diaph.#	Ht ft	Fx kip	Fy kip	X ft	Y ft
Roof	1	42.17	0.00	35.63	60.01	34.67
Level 3	1	27.50	0.00	103.16	62.07	36.94
Level 2	1	13.75	0.00	50.31	56.01	37.95

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kip	Sum Fy kip
Roof	1	0.000	35.631
Level 3	1	0.000	103.157
Level 2	1	0.000	50.315
		<hr/> 0.00	<hr/> 189.10

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Ht ft	Fx kip	Fy kip
Roof	42.17	0.00	35.63
Level 3	27.50	0.00	103.16
Level 2	13.75	0.00	50.31
		<hr/> 0.00	<hr/> 189.10

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RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Drift

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Steel Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects

Member Force Output: At Face of Joint

P-Delta: Yes Scale Factor (DL): 1.20 Scale Factor (LL): 1.00
 Scale Factor (Roof): 1.00 Scale Factor (Snow): 1.00

Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
E1	EQ	EQ_ASCE716_X_+E_F
E2	EQ	EQ_ASCE716_X_-E_F
E3	EQ	EQ_ASCE716_Y_+E_F
E4	EQ	EQ_ASCE716_Y_-E_F

Displacements for semirigid diaphragm are reported based on nodal displacements near to selected point.**Displacements for pseudo-flexible and flexible/none diaphragms are reported based on maximum nodal displacement within diaphragm boundary.**

RESULTS:

Location (ft): (118.000, 69.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	-0.1119	-0.1361	-0.0571	-0.1345	0.0003	0.0008
	Lp	-0.0493	-0.0235	-0.0192	-0.0251	0.0001	0.0001
	Ln	0.0013	-0.0003	0.0007	-0.0004	0.0000	0.0000
	E1	1.3507	-0.0126	0.3029	-0.0169	0.0017	0.0001
	E2	1.3694	-0.0478	0.3319	-0.0665	0.0019	0.0004
	E3	0.0315	0.2687	0.0461	0.0891	0.0003	0.0005
	E4	-0.0407	0.3280	-0.0640	0.2059	0.0004	0.0012
Level 3	D	-0.0548	-0.0016	-0.0259	-0.0006	0.0002	0.0000
	Lp	-0.0300	0.0016	-0.0147	0.0020	0.0001	0.0000
	Ln	0.0006	0.0001	0.0003	0.0001	0.0000	0.0000
	E1	1.0478	0.0042	0.3525	0.0053	0.0021	0.0000
	E2	1.0375	0.0187	0.3479	0.0116	0.0021	0.0001
	E3	-0.0145	0.1796	-0.0080	0.0868	0.0000	0.0005
	E4	0.0233	0.1221	0.0083	0.0621	0.0001	0.0004

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Drift

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Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
Level 2	D	-0.0288	-0.0010	-0.0288	-0.0010	0.0002	0.0000
	Lp	-0.0154	-0.0004	-0.0154	-0.0004	0.0001	0.0000
	Ln	0.0004	0.0000	0.0004	0.0000	0.0000	0.0000
	E1	0.6953	-0.0011	0.6953	-0.0011	0.0042	0.0000
	E2	0.6897	0.0071	0.6897	0.0071	0.0042	0.0000
	E3	-0.0066	0.0928	-0.0066	0.0928	0.0000	0.0006
	E4	0.0150	0.0600	0.0150	0.0600	0.0001	0.0004

Location (ft): (118.000, 3.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	-0.1119	-0.1361	-0.0505	-0.1345	0.0003	0.0008
	Lp	-0.0493	-0.0235	-0.0180	-0.0253	0.0001	0.0001
	Ln	0.0013	-0.0003	0.0005	-0.0004	0.0000	0.0000
	E1	1.3507	-0.0126	0.2954	-0.0165	0.0017	0.0001
	E2	1.3694	-0.0478	0.3014	-0.0657	0.0017	0.0004
	E3	0.0315	0.2687	0.0118	0.0933	0.0001	0.0005
	E4	-0.0407	0.3280	-0.0122	0.2085	0.0001	0.0012
Level 3	D	-0.0613	-0.0016	-0.0309	-0.0004	0.0002	0.0000
	Lp	-0.0313	0.0018	-0.0152	0.0023	0.0001	0.0000
	Ln	0.0008	0.0001	0.0004	0.0001	0.0000	0.0000
	E1	1.0553	0.0038	0.3616	0.0045	0.0022	0.0000
	E2	1.0681	0.0179	0.3674	0.0106	0.0022	0.0001
	E3	0.0197	0.1754	0.0105	0.0839	0.0001	0.0005
	E4	-0.0284	0.1195	-0.0107	0.0601	0.0001	0.0004
Level 2	D	-0.0304	-0.0012	-0.0304	-0.0012	0.0002	0.0000
	Lp	-0.0160	-0.0005	-0.0160	-0.0005	0.0001	0.0000
	Ln	0.0004	0.0000	0.0004	0.0000	0.0000	0.0000
	E1	0.6937	-0.0006	0.6937	-0.0006	0.0042	0.0000
	E2	0.7007	0.0074	0.7007	0.0074	0.0042	0.0000
	E3	0.0092	0.0915	0.0092	0.0915	0.0001	0.0006
	E4	-0.0178	0.0594	-0.0178	0.0594	0.0001	0.0004

Location (ft): (0.000, 69.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	-0.1119	-0.1361	-0.0575	-0.1466	0.0003	0.0008
	Lp	-0.0493	-0.0235	-0.0196	-0.0276	0.0001	0.0002
	Ln	0.0013	-0.0003	0.0007	-0.0002	0.0000	0.0000

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Drift

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Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	E1	1.3507	-0.0126	0.3005	-0.0080	0.0017	0.0000
	E2	1.3694	-0.0478	0.3296	-0.0184	0.0019	0.0001
	E3	0.0315	0.2687	0.0423	0.1435	0.0002	0.0008
	E4	-0.0407	0.3280	-0.0680	0.1065	0.0004	0.0006
Level 3	D	-0.0544	0.0105	-0.0251	0.0091	0.0002	0.0001
	Lp	-0.0296	0.0041	-0.0140	0.0034	0.0001	0.0000
	Ln	0.0006	-0.0002	0.0003	-0.0001	0.0000	0.0000
	E1	1.0502	-0.0047	0.3540	-0.0072	0.0021	0.0000
	E2	1.0398	-0.0294	0.3494	-0.0181	0.0021	0.0001
	E3	-0.0108	0.1252	-0.0049	0.0597	0.0000	0.0004
	E4	0.0274	0.2215	0.0113	0.1011	0.0001	0.0006
Level 2	D	-0.0292	0.0014	-0.0292	0.0014	0.0002	0.0000
	Lp	-0.0156	0.0006	-0.0156	0.0006	0.0001	0.0000
	Ln	0.0004	-0.0000	0.0004	-0.0000	0.0000	0.0000
	E1	0.6962	0.0025	0.6962	0.0025	0.0042	0.0000
	E2	0.6904	-0.0113	0.6904	-0.0113	0.0042	0.0001
	E3	-0.0059	0.0655	-0.0059	0.0655	0.0000	0.0004
	E4	0.0161	0.1204	0.0161	0.1204	0.0001	0.0007

Location (ft): (0.000, 3.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	-0.1119	-0.1361	-0.0496	-0.1463	0.0003	0.0008
	Lp	-0.0493	-0.0235	-0.0180	-0.0273	0.0001	0.0002
	Ln	0.0013	-0.0003	0.0006	-0.0002	0.0000	0.0000
	E1	1.3507	-0.0126	0.2997	-0.0097	0.0017	0.0001
	E2	1.3694	-0.0478	0.3055	-0.0203	0.0017	0.0001
	E3	0.0315	0.2687	0.0160	0.1412	0.0001	0.0008
	E4	-0.0407	0.3280	-0.0078	0.1033	0.0000	0.0006
Level 3	D	-0.0623	0.0102	-0.0322	0.0082	0.0002	0.0000
	Lp	-0.0313	0.0038	-0.0150	0.0028	0.0001	0.0000
	Ln	0.0008	-0.0002	0.0004	-0.0001	0.0000	0.0000
	E1	1.0510	-0.0029	0.3593	-0.0065	0.0022	0.0000
	E2	1.0640	-0.0276	0.3649	-0.0172	0.0022	0.0001
	E3	0.0155	0.1276	0.0074	0.0608	0.0000	0.0004
	E4	-0.0329	0.2247	-0.0129	0.1022	0.0001	0.0006
Level 2	D	-0.0301	0.0019	-0.0301	0.0019	0.0002	0.0000
	Lp	-0.0162	0.0010	-0.0162	0.0010	0.0001	0.0000
	Ln	0.0004	-0.0000	0.0004	-0.0000	0.0000	0.0000

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Drift

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Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	E1	0.6917	0.0036	0.6917	0.0036	0.0042	0.0000
	E2	0.6990	-0.0103	0.6990	-0.0103	0.0042	0.0001
	E3	0.0081	0.0667	0.0081	0.0667	0.0000	0.0004
	E4	-0.0200	0.1225	-0.0200	0.1225	0.0001	0.0007

TORSIONAL IRREGULARITY DATA:**X-Axis:**

Story	LdC	Drift in	Coord ft	Drift in	Coord ft	Max/Min	Max/Ave
Roof	E2	0.3014	(118.00 3.00)	0.3319	(118.00 69.00)	1.101	1.048
Level 3	E2	0.3674	(118.00 3.00)	0.3479	(118.00 69.00)	1.056	1.027
Level 2	E2	0.7007	(118.00 3.00)	0.6897	(118.00 69.00)	1.016	1.008

Y-Axis:

Story	LdC	Drift in	Coord ft	Drift in	Coord ft	Max/Min	Max/Ave
Roof	E4	0.1033	(0.00 3.00)	0.2085	(118.00 3.00)	2.019	1.337
Level 3	E4	0.1022	(0.00 3.00)	0.0601	(118.00 3.00)	1.702	1.260
Level 2	E4	0.1225	(0.00 3.00)	0.0594	(118.00 3.00)	2.061	1.347

Max Drift Satisfy
Limite

Max/Ave>1.2
Torsional Irregularity

LOAD CASE: Winds

ASCE 7-16

Exposure: B

Basic Wind Speed (mph): 114.0

Apply Directionality Factor, $K_d = 0.85$ Use Topography Factor, $K_{zt} = 1.00$ Ground Elevation Factor, $K_e = 1.00$

Use Calculated Frequency for X-Dir.

Use Calculated Frequency for Y-Dir.

Gust Factor for Flexible Structures, G : Use Calculated G for X-Dir.Gust Factor for Rigid Structures, G : Use Calculated G for Y-Dir.

Damping Ratio for Flexible Structures = 0.01

Mean Roof Height (ft): Top Story Height + Parapet = 42.17

Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 0.729 Structure is Flexible

Y-Direction: Natural Frequency = 2.667 Structure is Rigid

 C_p Windward = 0.80 q Leeward (q_h) = 21.84 psf $G C_{pn}$ (Parapet): Windward = 1.50 Leeward = -1.00

Height ft	K_z	K_{zt}	q_z psf	Gust Factor G		C_p Leeward		Pressure (psf)	
				X	Y	X	Y	X	Y
42.17	0.772	1.000	21.836	1.008	0.820	-0.289	-0.500	23.980	23.274
27.50	0.683	1.000	19.326	0.997	0.820	-0.298	-0.500	21.900	21.627
13.75	0.575	1.000	16.253	0.997	0.820	-0.298	-0.500	19.449	19.612
0.00	0.575	1.000	16.253	0.997	0.820	-0.298	-0.500	19.449	19.612

Windward and Leeward Wind Pressures for Semirigid Diaphragms:

Story	Diaph. #	Windward Pressure (psf)		Leeward Pressure (psf)	
		X	Y	X	Y
Level 3	1	15.410	12.676	6.490	8.952
Level 2	1	12.959	10.660	6.490	8.952

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_X

Level	Diaph.#	Ht ft	F_x kips	F_y kips	X ft	Y ft
Roof	1	42.17	12.27	0.00	68.75	34.92
Level 3	1	27.50	23.26	0.00	68.75	36.50
Level 2	1	13.75	21.39	0.00	68.75	38.17

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum F_x kips	Sum F_y kips
Roof	1	12.267	0.000

RAM Structural System

Bentley

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

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Loads and Applied Forces

Level 3	1	23.256	0.000
Level 2	1	21.388	0.000
		<hr/> 56.91	<hr/> 0.00

APPLIED STORY FORCES

Type: Wind_ASCE716_1_X

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	12.27	0.00
Level 3	27.50	23.26	0.00
Level 2	13.75	21.39	0.00
		<hr/> 56.91	<hr/> 0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	0.00	26.58	68.75	34.92
Level 3	1	27.50	0.00	48.92	68.75	38.17
Level 2	1	13.75	0.00	43.75	68.75	38.17

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	0.000	26.581
Level 3	1	0.000	55.129
Level 2	1	0.000	47.656
		<hr/> 0.00	<hr/> 129.37

APPLIED STORY FORCES

Type: Wind_ASCE716_1_Y

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	0.00	26.58
Level 3	27.50	0.00	48.92
Level 2	13.75	0.00	43.75
		<hr/> 0.00	<hr/> 119.25

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RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

Building Code: IBC

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Steel Code: AISC360-16 LRFD

CRITERIA:

Rigid End Zones:	Ignore Effects
Member Force Output:	At Face of Joint
P-Delta:	Yes
Scale Factor (DL):	1.20
Scale Factor (LL):	1.00
Scale Factor (Roof):	1.00
Scale Factor (Snow):	1.00

Ground Level: Base

Mesh Criteria :

Max. Distance Between Nodes on Mesh Line (ft) : 4.00

Merge Node Tolerance (in) : 0.0150

Geometry Tolerance (in) : 0.0100

Walls Out-of-plane Stiffness Not Included in Analysis.

Sign considered for Dynamic Load Case Results.

Rigid Links Included at Fixed Beam-to-Wall Locations

Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

NOTE:

MOMENT FRAME CHECK AND
BRACED FRAME CHECK ARE BOTH
INCLUDED. SEE NEXT A FEW PAGES
FOR INDICATION.

LOAD COMBINATION CRITERIA:

Live Load factor f1 (0.5 or 1.0)	0.500
Sds (for Ev)	0.434
RhoX	1.000
RhoY	1.000

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
E1	EQ	EQ_ASCE716_X_+E_F
E2	EQ	EQ_ASCE716_X_-E_F
E3	EQ	EQ_ASCE716_Y_+E_F
E4	EQ	EQ_ASCE716_Y_-E_F
E5	EQ	EQ_ASCE716_X_+E_0.3Y_+E_F
E6	EQ	EQ_ASCE716_X_+E_-0.3Y_+E_F
E7	EQ	EQ_ASCE716_-X_+E_0.3Y_+E_F
E8	EQ	EQ_ASCE716_-X_+E_-0.3Y_+E_F
E9	EQ	EQ_ASCE716_0.3X_+E_Y_+E_F
E10	EQ	EQ_ASCE716_0.3X_+E_-Y_+E_F
E11	EQ	EQ_ASCE716_-0.3X_+E_Y_+E_F
E12	EQ	EQ_ASCE716_-0.3X_+E_-Y_+E_F
E13	EQ	EQ_ASCE716_X_+E_0.3Y_-E_F
E14	EQ	EQ_ASCE716_X_+E_-0.3Y_-E_F
E15	EQ	EQ_ASCE716_-X_+E_0.3Y_-E_F
E16	EQ	EQ_ASCE716_-X_+E_-0.3Y_-E_F
E17	EQ	EQ_ASCE716_0.3X_+E_Y_-E_F
E18	EQ	EQ_ASCE716_0.3X_+E_-Y_-E_F
E19	EQ	EQ_ASCE716_-0.3X_+E_Y_-E_F
E20	EQ	EQ_ASCE716_-0.3X_+E_-Y_-E_F
E21	EQ	EQ_ASCE716_X_-E_0.3Y_+E_F
E22	EQ	EQ_ASCE716_X_-E_-0.3Y_+E_F

RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

Building Code: IBC

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Steel Code: AISC360-16 LRFD

387 * 0.813 D - 1.000 E36

* = Load combination currently selected to use

SAMPLE BRACED FRAME
SECTION STEEL
STANDARD CODE CHECKSOME LOAD
COMBINATION NOT SHOWN
FOR CLARITY

Frame #1:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
12	14.69	0.00	0.00	-0.30	-0.00	1	0.02 Axial	50	W14X68
13	15.66	0.00	0.00	-0.30	-0.00	1	0.03 Axial	50	W14X68

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
31	3.14	11.29	0.00	-3.36	-0.00	133	0.01 H1-1b	50	W24X131

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
22	12.99	0.00	0.00	0.00	0.00	205	0.60 Axial	50	HSS3.5X3.5X5/16
23	13.19	0.00	0.00	-0.00	0.00	134	0.61 Axial	50	HSS3.5X3.5X5/16

Level: Level 3

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
13	102.74	0.00	0.00	0.28	-0.18	133	0.16 Axial	50	W14X68
14	119.93	0.00	0.00	0.28	-0.18	134	0.19 Axial	50	W14X68

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
35	-2.41	196.47	0.01	-27.39	0.01	2	0.36 H1-1b	50	W18X71

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
18	55.72	0.00	0.00	-0.00	-0.00	134	0.47 Axial	50	HSS5.5X5.5X3/8
19	51.34	0.00	0.00	-0.00	-0.00	133	0.43 Axial	50	HSS5.5X5.5X3/8

Level: Level 2

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
13	144.34	1.23	1.52	-0.12	-0.05	133	0.24 H1-1a	50	W14X68
14	172.84	0.25	-1.57	-0.13	-0.10	134	0.28 H1-1a	50	W14X68

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
36	-0.48	-44.83	-0.01	-16.84	-0.00	2	0.08 H1-1b	50	W18X71

RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

Building Code: IBC

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Steel Code: AISC360-16 LRFD

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
10	90.15	0.00	0.00	0.00	0.00	133	0.76 Axial	50	HSS5.5X5.5X3/8
11	92.58	0.00	0.00	-0.00	0.00	134	0.78 Axial	50	HSS5.5X5.5X3/8

Frame #2:

SAMPLE MOMENT FRAME
SECTION STEEL
STANDARD CODE CHECK

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
18	15.44	26.52	-0.00	-2.44	-0.00	106	0.05 H1-1b	50	W18X86
35	14.05	41.14	0.00	-3.28	0.00	115	0.04 H1-1b	50	W18X130
36	9.97	35.72	-0.00	3.51	-0.00	106	0.06 H1-3a(H1-1b)	50	W18X86

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
126	1.90	-28.96	0.00	-5.14	0.00	184	0.07 H1-3a(H1-1b)	50	W14X68
127	3.49	-35.61	0.00	6.67	-0.00	115	0.08 H1-3a(H1-1b)	50	W14X68

Level: Level 3

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
20	80.38	105.30	-0.00	-12.84	0.02	115	0.19 H1-3a(H1-1b)	50	W18X86
44	65.86	163.05	0.00	-20.65	0.02	115	0.17 H1-3a(H1-1b)	50	W18X130
45	65.29	-95.59	-0.00	11.54	0.02	112	0.17 H1-3a(H1-1b)	50	W18X86

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
140	4.99	-123.16	-0.06	-22.63	0.05	112	0.23 H1-3a(H1-1b)	50	W21X62
141	5.94	-153.29	-0.10	31.46	-0.02	106	0.29 H1-3a(H1-1b)	50	W21X62

Level: Level 2

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
26	111.07	231.88	0.16	-17.41	0.01	115	0.25 H1-3a(H1-1b)	50	W18X130
32	120.09	-132.11	0.12	9.81	-0.02	112	0.24 H1-3a(H1-1b)	50	W18X86

RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

Building Code: IBC

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Steel Code: AISC360-16 LRFD

Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
48	150.06	139.59	0.23	-10.21	0.00	115	0.27 H1-3a(H1-1b)	50	W18X86

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
71	-1.72	-223.01	0.01	35.39	0.01	115	0.41 H1-1b	50	W21X62
149	-0.93	-192.42	0.01	-29.00	-0.00	115	0.36 H1-1b	50	W21X62

MOMENT FRAME DESIGN

Seismic Provisions Member Code Check Summary

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Frame #2:

Story:Roof Column: 18 Type: Special Moment Resisting Frame

Size:W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Roof Column: 35 Type: Special Moment Resisting Frame

Size:W18X130 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Roof Column: 36 Type: Special Moment Resisting Frame

Size:W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Roof Beam:126 Type: Special Moment Resisting Frame

Size:W14X68 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story:Roof Beam:127 Type: Special Moment Resisting Frame

Size:W14X68 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story:Level 3 Column: 20 Type: Special Moment Resisting Frame

Size:W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Level 3 Column: 44 Type: Special Moment Resisting Frame

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Size: W18X130 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 3**Column: 45****Type: Special Moment Resisting Frame**

Size: W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 3**Beam: 140****Type: Special Moment Resisting Frame**

Size: W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story: Level 3**Beam: 141****Type: Special Moment Resisting Frame**

Size: W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story: Level 2**Column: 26****Type: Special Moment Resisting Frame**

Size: W18X130 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 2**Column: 32****Type: Special Moment Resisting Frame**

Size: W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 2**Column: 48****Type: Special Moment Resisting Frame**

Size: W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

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RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Level 2 Beam: 71 Type: Special Moment Resisting Frame

Size:W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story:Level 2 Beam:149 Type: Special Moment Resisting Frame

Size:W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

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RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Seismic Provisions Member Code Check

SAMPLE BEAM CALC.

02/18/21 15:38:14

Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Level 2 Frame No: 2 Member No: 71

Fy (ksi): 50.00 Size: W21X62

Frame Type: Special Moment Resisting Frame

Apply AISC 358 Provisions to Moment Frame

E3.4b Stability Bracing of Beams (D1.2b Highly Ductile) --- OK

Max Lu (in) = 72.00 Lu Limit = 88.79 OK

Lateral Bracing Requirements along Beam

Required strength of lateral brace along beam (A-6-7) = 7.77 kip

Required stiffness of bracing (A-6-8) = 5180.28 kip / Lb

Lateral Bracing Requirements at Plastic Hinge

Required strength of lateral brace at hinge = 23.31 kip

Required stiffness of bracing (A-6-8) = 5180.28 kip / Lb

 $Cd = 1.0$ assumed for eqns (A-6-7/8) Lb = distance between braces (in)*The brace closest to the point of inflection should be designed for two times these values ($Cd=2$)***E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK**Flange b/t_f = 6.70 Limit = 7.35 OKWeb h/t_w = 46.90 Limit = 59.01 OK**E3.6b Beam-to-Column Connection Requirements***Beam-to-column connection must be capable of sustaining a 0.04 radian interstory drift ratio.**Required flexural strength of connection (kip-ft) = $0.8M_p$ = 480.00 at story drift angle in (1)**All beam-to-column joints to demonstrate conformance with E3.6b(1) as indicated in E3.6b(2)**See Additional AISC 358 Requirements for Assigned Connection Type.*This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v3B>

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Seismic Provisions Member Code Check

SAMPLE COLUMN CALC.

02/18/21 15:38:14

Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Level 2 Frame No: 2 Member No: 32
 Fy (ksi): 50.00 Size: W18X86
 Frame Type: Special Moment Resisting Frame
 Apply AISC 358 Provisions to Moment Frame

D1.4a Required Strength --- OK

Compression: Max Pu (kip) = 162.38 --- Combination: 1.287 D + 0.500 Lp - 3.000 E16
 Max Pu/φPn = 0.19 **OK**
Tension: Max Pu (kip) = 9.27 --- Combination: 0.813 D - 3.000 E13
 Max Pu/φPn = 0.01 **OK**

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

Required tension and compression strength from D1.4a.

Shear in major axis (kip) = 27.06 --- Combination: 1.287 D + 0.500 Lp + 3.000 E13

Shear in minor axis (kip) = 0.15 --- Combination: 1.287 D + 0.500 Lp - 3.000 E28

Moment in major axis (kip-ft) = 366.75 --- Combination: 1.287 D + 0.500 Lp - 3.000 E16

Moment in minor axis (kip-ft) = 0.65 --- Combination: 1.287 D + 0.500 Lp + 3.000 E10

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

See Additional AISC 358 Requirements for Assigned Connection Type.

E3.4c(1) Unbraced Connections --- OK

Joint Above Column is Restrained

Joint Below Column is Restrained

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

For calculation of web limit: Pu (kip) = 162.38 φPy (kip) = 1138.50

Flange b/tf = 7.21 Limit = 7.35 **OK**

Web h/tw = 33.46 Limit = 51.27 **OK**

E3.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

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RAM Structural System

Seismic Provisions Joint Code Check

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Joint Parameters

Story: Level 2 Frame No: 2 Joint No: 45
 Fy (ksi): 50.00 Column Size: W18X130
 Joint Frame Type: Special Moment Resisting Frame
 Apply AISC 358 Provisions to Moment Frame
 Strain Hardening Factor Cpr = 1.40

E3.4a Moment Ratio --- OK

Col.	Size	M*pc (kip-ft)	Zc (in ³)	Pu (kip)	Combo#
Top	W18X130	1161.45	290.00	74.11	13
Bot	W18X130	1123.62	290.00	133.91	13
Bm.	Size	M*pb (kip-ft)	Mpr (kip-ft)	Muv (kip-ft)	Sh+dc/2 (in)
71	W21X62	1011.64	924.00	87.64	9.65
149	W21X62	1000.30	924.00	76.30	9.65

Sh = Distance face-of-column to location of hinge.

Mpr = *Cpr* x *Ry* x *Fy* x *Zhng*, where *Zhng* = *Z* at hinge.

$\Sigma M^*_{pc} = \Sigma Z_c (F_y - P_u / A_g)$ (kip-ft) = 2285.07 ΣM^*_{pb} (kip-ft) = 2011.94

$\Sigma M^*_{pc} / \Sigma M^*_{pb}$ (Eqn E3-1) = 1.14 > 1.0 OK

E3.4c Stability Bracing at Beam-to-Column Connections --- OK

Joint is restrained in the minor axis at both the top and bottom flange of the moment-frame beams.

Note! RAM Frame assumes that a beam framing into the minor axis of the joint, braces the column at the elevation of both top and bottom flange of the girders connected to the flanges of the column.

E3.6d Connection Required Shear Strength

Bm No.	Col Side	Mpr (kip-ft)	L (ft)	0.8Mp(kip-ft)	V Max (kip)
71	Flange	924.00	20.429	480.00	108.98
149	Flange	924.00	17.429	480.00	117.18

V Max = Max shear from applicable load combinations with *Ecl*

Mpr = *Cpr* x *Ry* x *Fy* x *Zhng*, where *Zhng* = *Z* at hinge.

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- OK With Web Plate / Stiffeners

Bm.	Size	Bf Red. (in)	Z (in ³)	Z hng (in ³)	Mpr (kip-ft)
71	W21X62	0.000	144.00	144.00	924.00
149	W21X62	0.000	144.00	144.00	924.00
Bm.	Mpr (kip-ft)	Sh (in)	V hng (kip)	Mf (kip-ft)	Vpz (kip)
71	924.00	0.00	108.978	924.00	543.93
149	924.00	0.00	117.182	924.00	543.93

Sh = Distance face-of-column to location of hinge.

Vhng = Shear at hinge or col face from applicable load combinations with *Ecl*

Mpr = *Cpr* x *Ry* x *Fy* x *Zhng*, where *Zhng* = *Z* at hinge.

Required Panel Zone Shear Strength (kip) = 953.46

Panel Zone Shear Reduced by *Vc* (kip) = 134.40

Axial Comp. for Panel Zone Calc. (kip) = 133.91 - Combination 1.287 D + 0.500 Lp - 3.000 E16

Panel Zone Strength Without Web Plate (kip) = 457.05 NG

Web Plate Reqd Thickness (in) = 0.857

Web Plate Thickness Used (in) = 0.875

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RAM Structural System

Seismic Provisions Joint Code Check

RAM Structural System 17.01.01.05

DataBase: GRCC St

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Steel Code: AISC341-16 - LRFD

Web plate thickness req'd to fillet weld to column flanges (in) = 0.212

Panel Zone Strength With Web Plate (kip) = 963.68 **OK**

E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- **OK**

No plug welds specified so both column and web plate must meet Equation E3-7

dz (in) = 19.770 wz (in) = 16.900

Column web thickness reqd (in) = 0.407 Thickness provided = 0.670 **OK**Web PL thickness reqd (in) = 0.407 Thickness provided = 0.875 **OK**

358 - 2.4.4 Continuity Plates --- **OK With Web Plate / Stiffeners**

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
71	W21X62	50.00	8.24	0.615	1.37	1.20	Yes
149	W21X62	50.00	8.24	0.615	1.37	1.20	Yes

See Additional AISC 358 Requirements for Assigned Connection Type.

BRACED FRAME DESIGN

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Criteria

Use frame numbers to designate a single line of bracing

 $C_d/I_e = 4.0$

Apply AISC 358 Provisions to Moment Frame

RBS Strain Hardening Factor $C_{pr} = 1.10$ Other Strain Hardening Factor $C_{pr} = 1.40$

AISC 358 Live Load Factor = 0.50

AISC 358 Snow Load Factor = 0.20

BRBF and SCBF - Required Column Strength based on Larger of Strength and Capacity Limited Seismic Forces.

SAMPLE BRACED FRAME

Frame #1:

Story:Roof **Column: 12** **Type: Special Concentric Brace Frame -Chevron**Size:W14X68 F_y (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story:Roof **Column: 13** **Type: Special Concentric Brace Frame -Chevron**Size:W14X68 F_y (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story:Roof **Beam: 31** **Type: Special Concentric Brace Frame -Chevron**Size:W24X131 F_y (ksi): 50.00

F2.4b V- and Inverted V- Braced Frames --- Additional Check Required

F2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Story:Roof **Brace: 22** **Type: Special Concentric Brace Frame -Chevron**Size:HSS3.5X3.5X5/16 F_y (ksi): 50.00

F2.4a Lateral Force Distribution --- OK

F2.5 Bracing Members --- OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

F2.5b(1) Slenderness --- OK

Story:Roof **Brace: 23** **Type: Special Concentric Brace Frame -Chevron**Size:HSS3.5X3.5X5/16 F_y (ksi): 50.00

F2.4a Lateral Force Distribution --- OK

F2.5 Bracing Members --- OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

F2.5b(1) Slenderness --- OK

Story:Level 3 **Column: 13** **Type: Special Concentric Brace Frame -Chevron**

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Size: W14X68 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 3**Column: 14****Type: Special Concentric Brace Frame -Chevron**

Size: W14X68 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 3**Beam: 35****Type: Special Concentrically Braced Frame**

Size: W18X71 Fy (ksi): 50.00

F2.3 Non-V and Non-Inverted V-Braced Frame - Unbalanced Forces Design Required.

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 3**Brace: 18****Type: Special Concentric Brace Frame -Chevron**

Size: HSS5.5X5.5X3/8 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- OK

F2.5 Bracing Members --- OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

F2.5b(1) Slenderness --- OK

Story: Level 3**Brace: 19****Type: Special Concentric Brace Frame -Chevron**

Size: HSS5.5X5.5X3/8 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- OK

F2.5 Bracing Members --- OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

F2.5b(1) Slenderness --- OK

Story: Level 2**Column: 13****Type: Special Concentric Brace Frame -Chevron**

Size: W14X68 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 2**Column: 14****Type: Special Concentric Brace Frame -Chevron**

Size: W14X68 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Story:Level 2 Beam: 36 Type: Special Concentric Brace Frame -Chevron

Size:W18X71 Fy (ksi): 50.00

F2.4b V- and Inverted V- Braced Frames --- **Additional Check Required**F2.5a Basic Requirements (D1.1 Moderately Ductile) --- **OK****Story:Level 2 Brace: 10 Type: Special Concentric Brace Frame -Chevron**

Size:HSS5.5X5.5X3/8 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- **OK**F2.5 Bracing Members --- **OK**F2.5a Basic Requirements (D1.1 Highly Ductile) --- **OK**F2.5b(1) Slenderness --- **OK****Story:Level 2 Brace: 11 Type: Special Concentric Brace Frame -Chevron**

Size:HSS5.5X5.5X3/8 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- **OK**F2.5 Bracing Members --- **OK**F2.5a Basic Requirements (D1.1 Highly Ductile) --- **OK**F2.5b(1) Slenderness --- **OK**

ADDITIONAL CHECK WILL
BE SHOWN IN BRACED
FRAME MEMBER CHECK
SAMPLE CALC.

RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Level 2

Frame No: 1

Member No: 36

Fy (ksi): 50.00

Size: W18X71

Frame Type: Special Concentric Brace Frame -Chevron

Criteria

Use frame numbers to designate a single line of bracing

F2.4b V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

Top Left Brace: Tens (kip) = 404.54

Comp (kip) = 149.61

Post Buck. Comp (kip) = 44.88

Top Right Brace: Tens (kip) = 404.54

Comp (kip) = 149.61

Post Buck. Comp (kip) = 44.88

Bot Left Brace: Tens (kip) = 404.54

Comp (kip) = 149.61

Post Buck. Comp (kip) = 44.88

Bot Right Brace: Tens (kip) = 404.54

Comp (kip) = 149.61

Post Buck. Comp (kip) = 44.88

Unbalanced Beam Forces from V-Braces - F2.3(a) Analysis

Vertical Resultant (kip) = 0.0

Horizontal Resultant (kip) = 0.0

Unbalanced Beam Forces from V-Braces - F2.3(b) Analysis

Vertical Resultant (kip) = 0.0

Horizontal Resultant (kip) = 0.0

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 86.00 Lu Limit = 170.58 **OK**

Required strength of lateral brace along beam (A-6-7) = 0.81 kip

Required stiffness of bracing (A-6-8) = 543.06 kip / Lb

*Cd = 1.0 assumed for eqns (A-6-7/8)**Lb = distance between braces (in)**The brace closest to the point of inflection should be designed for two times these values (Cd=2)*

F2.5a Basic Requirements (D1.1 Moderately Ductile) --- **OK**

Flange b/tf = 4.72 Limit = 9.18 **OK**Web h/tw = 32.48 Limit = 36.05 **OK***Note: Beam axial load from F2.3 Analysis not determined; Web h/tw limit assumed to be $1.57 \sqrt{E/R_y F_y}$.*This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v3B>

SCBF Beam Additional Check

Per AISC Seismic Provision Section F2.3(i) and F2.3(ii)

Roof Beam

Try W24X131	$L_p := 29.33 \text{ ft}$	
$P_n := 1624.91$	Kips	$P_u1 := 3.5$
$M_{nx} := 1387.5$	Kips - ft	$M_u1 := 10.5$
$M_{ny} := 305.6$	Kips - ft	

F2.3(a)

With Unbalanced Load:

Vertical Resultant		Horizontal Resultant
$E_{mv} := 128.3$	Kips	$E_{mh} := 163.9$ Kips

Axial Load

$$P_u := P_{u1} + E_{mh} = 167.4 \text{ Kips}$$

Moment:

$$M_{ux} := M_{u1} + E_{mv} \cdot \frac{L}{4} = 951.26 \text{ Kips - ft}$$

$$M_{uy} := 0 \text{ Kips - ft}$$

Check P_u/P_n

$$\frac{P_u}{P_n} = 0.103$$

$P_r/P_c > 0.2$ Use Equation

$$\frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad H1 - 1a$$

$P_r/P_c < 0.2$ Use Equation

$$\frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad H1 - 1b$$

Interaction Check

$$\text{Int_Check} := \begin{cases} \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} > 0.2 \\ \frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} \leq 0.2 \end{cases}$$

$$\text{Int_Check} = 0.737$$

F2.3(b)

With Unbalanced Load:

Vertical Resultant

$$E_{mv} := 142.1 \text{ Kips}$$

Horizontal Resultant

$$E_{mh} := 150.4 \text{ Kips}$$

Axial Load

$$P_u := P_{u1} + E_{mh} = 153.9 \text{ Kips}$$

Moment:

$$M_{ux} := M_{u1} + E_{mv} \cdot \frac{L}{4} = 1.052 \times 10^3 \text{ Kips} \cdot \text{ft}$$

$$M_{uy} := 0 \text{ Kips} \cdot \text{ft}$$

Check P_u/P_n

$$\frac{P_u}{P_n} = 0.095$$

$P_r/P_c > 0.2$ Use Equation

$$\frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad \text{H1 - 1a}$$

Interaction Check

$$\text{Int_Check} := \begin{cases} \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} > 0.2 \\ \frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{otherwise} \end{cases}$$

$P_r/P_c < 0.2$ Use Equation

$$\frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad \text{H1 - 1b}$$

$$\text{Int_Check} = 0.806$$

W24X131 OK

SCBF Beam Additional Check

Per AISC Seismic Provision Section F2.3(i) and F2.3(ii)

Third Floor Beam

Try W18x71 $L := 29.3$ ft

$P_n := 936$	Kips	$P_{u1} := 12$
$M_{nx} := 547.5$	Kips - ft	$M_{u1} := 376$
$M_{ny} := 92.63$	Kips - ft	

F2.3(b) By inspection, we know this will govern

With Unbalanced Load:

From Member Force in Ram,

$F_{upper} := 206.98$ Kips For Expected Tension of Brace 3.5x3.5x5/16

$F_{lower} := 59.71$ Kips For Expected Post Buckling Comp of Brace 6 x 6 x 3/8

$F_{Diaphragm} := 60$ Kips From Ram Structure, Frame Story Shear

$\theta_1 := 44.705$

$\theta_2 := 43.288$

$$E_{mh} := F_{upper} \cdot \cos(\theta_1 \text{ deg}) - F_{lower} \cdot \cos(\theta_2 \text{ deg}) + F_{Diaphragm} = 163.645 \text{ Kips}$$

Vertical Resultant

$$E_{mv} := 0$$

Kips

Horizontal Resultant

$$E_{mh} = 163.645 \text{ Kips}$$

Axial Load

$$P_u := P_{u1} + E_{mh} = 175.645 \text{ Kips}$$

Moment:

$$M_{ux} := M_{u1} + E_{mv} \cdot \frac{L}{4} = 376 \text{ Kips} \cdot \text{ft}$$

$$M_{uy} := 0 \text{ Kips} \cdot \text{ft}$$

Check P_u/P_n

$$\frac{P_u}{P_n} = 0.188$$

$P_r/P_c > 0.2$ Use Equation

$$\frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad \text{H1 - 1a}$$

Interaction Check

$$\text{Int_Check} := \begin{cases} \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} > 0.2 \\ \frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{otherwise} \end{cases}$$

$P_r/P_c < 0.2$ Use Equation

$$\frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad \text{H1 - 1b}$$

$$\text{Int_Check} = 0.781$$

W18x71 OK

Second Floor Beam

Since the Brace at top and bot are the same(Size and Geometry), Unblanced Load can be ignored, use W18x71

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Seismic Provisions Member Code Check

02/18/21 15:56:10

Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Level 2 Frame No: 1 Member No: 14
 Fy (ksi): 50.00 Size: W14X68
 Frame Type: Special Concentric Brace Frame -Chevron

Criteria

Use frame numbers to designate a single line of bracing

D1.4a Required Strength --- OK

Max of Adj. Brace Strength and Amp. Seismic Forces with Ω_o Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 499.37 --- Combination: 1.287 D + 0.500 Lp + 1.000 Ecl

Ecl (kips) = 363.94

Ecl: Controlled by F2.3(a) Analysis

Max Pu/ ϕ Pn = 0.77 OK

Tension: Max Pu (kip) = 112.79 --- Combination: 0.813 D + 1.000 Ecl

Ecl (kips) = -187.48

Ecl: Controlled by F2.3(a) Analysis

Max Pu/ ϕ Pn = 0.13 OK

Exception (2) of F2.3 was not checked.

D1.4a(b) – E amplified by Ω_o :

	Load Combination	Pu(kip)	ϕ Pn(kip)	Result
Compression:	1.287 D + 0.500 Lp - 3.000 E34	247.67	647.66	OK
Tension:	0.813 D - 3.000 E35	37.56	900.00	OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.287 D + 0.500 Lp + 1.000 Ecl	499.37
Tension:	0.813 D + 1.000 Ecl	112.79

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

For calculation of web limit: Pu (kip) = 499.37 ϕ Py (kip) = 900.00

Flange b/tf = 6.94 Limit = 7.35 OK

Web h/tw = 27.42 Limit = 42.94 OK

F2.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

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RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Seismic Provisions Member Code Check

02/18/21 15:56:10

Steel Code: AISC341-16 - LRFD

Brace Parameters

Story: Level 2 Frame No: 5 Member No: 15
 Fy (ksi): 42.00 Size: HSS5X5X3/8
 Frame Type: Special Concentric Brace Frame -Chevron

Criteria

Use frame numbers to designate a single line of bracing

F2.5 Bracing Members --- OK

F2.3 Required Strength

Expected tension strength ($R_y \times F_y \times A_g$) of brace (kip) = 363.38

Expected comp. str. min ($R_y F_y A_g, (1.0/0.877) F_{cre} A_g$) of brace (kip) = 106.72

Expected post-buckling strength of brace (kip) = 32.02

F2.4a Lateral Force Distribution --- OK

The max percentage of lateral load carried by braces for frame # 5 (this story only).

Max Horiz. Brace Force	Load Case	Percent	Result
Compression:	EQ_ASCE716_X_+E_0.3Y_-E	50.23	<70 OK
	F		
Tension:	EQ_ASCE716_X_+E_0.3Y_-E	50.23	<70 OK
	F		

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Flange b/t_f = 11.33 Limit = 14.44 OK

Web h/t_w = 11.33 Limit = 14.44 OK

F2.5b(1) Slenderness --- OK

Max kl/r = 128.74 kl/r limit = 200.00 OK

F2.6 Bracing Connections

F2.6c(1) Required Tensile Strength

Strength of the connection shall be the lesser of a or b as follows:

- Expected ($R_y \times F_y \times A_g$) tensile strength of brace (kip) = 363.38
- Max force that the brace can transfer to the system - Not Performed

F2.6c(2) Required Compressive Strength

Required compression strength of brace P_c (kip) = 106.72

F2.6c(3a) Rotation Capacity

Brace buckles in direction of member Minor axis, $1.1R_y M_p$ (kip-ft) = 57.13

The design of the gusset plates shall include consideration of buckling.

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Example Connection Calc

Bay Size

$L = 14.665$ ft
 $H_{lower} = 13.75$ ft
 $\theta_{lower} = 46.8443614$ deg.
 $H_{upper} = 13.75$ ft
 $\theta_{upper} = 43.1556386$ deg.

Members

Column = W14X68
 BM Stub = W18X71
 Beam = W18X71
 Slab Thk. = 5.5 in

Mat'l:

Gusset = A572-50
 Brace = A500-C SQ & RECT
 Reinf. PL = A572-50
 Beam = A992
 Column = A992
 Splice PL = A572-50

Lower Brace

HSS5X5X3/8
Gusset Details:
 $\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 2$ in.
 $t_g = 1$ in.
 $L_w = 18$ in.
 Pull-off Dim = 32.5 in.
 Clip = 0.75 in.
 Over-slot = 2 in.

Gusset Weld Details:

$D_{brace} = 5$
 $D_{column} = 5$
 $D_{beam} = 5$
 $D_{stub-col} = 5$
Reinforcing Details:
 $t_{PL} = 0.75$ in.
 $b = 4$ in.
 $L_w = 10$ in.
 $D_{brace} = 6$

Upper Brace

HSS3-1/2X3-1/2X5/16
Gusset Details:
 $\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 2$ in.
 $t_g = 1$ in.
 $L_w = 18$ in.
 Pull-off Dim = 28.5 in.
 Clip = 0.75 in.
 Over-slot = 2 in.

Gusset Weld Details:

$D_{brace} = 5$
 $D_{column} = 5$
 $D_{beam} = 5$
Reinforcing Details:
 $t_{PL} = 0.75$ in.
 $b = 2.5$ in.
 $L_w = 10$ in.
 $D_{brace} = 6$

Analysis

$P_{uc} = 38$ k
 Drag DF = 1
 $F_{EXX} = 70$ ksi
 Ductility Factor = 1.25
 Gusset K Factor = 0.6
 Distribution Method = UFM

Splice Plate

Axial Load = 276.19 k
 Grav. Shear = 27.5 k
 $t_{PL} = 0.625$ in
 Bolts (DS) = A325-X
 Dia. = 1 in
 Bolt Rows = 4
 Bolt Col. = 3
 Row Spacing = 3 in
 Col. Spacing = 3 in
 le = 2 in
 Gap = 1 in
 Dist = 27.3 in

Connection Transfer Forces

$A_I = 242.1123957$ k
 $N_I = 262.4563474$ k
 $A_{II} = 160.9249126$ k
 $N_{II} = 276.1939357$ k

Upper Gusset

Gusset Edge Forces

Sway Right

$R_y F_y A_g = 228.8$ k
 $H_{uc} = 58.4$ k
 $V_{uc} = 79.3$ k
 $M_{uc} = 89.9$ in-k
 $H_{ub} = 108.5$ k
 $V_{ub} = 77.2$ k
 $M_{ub} = 0.0$ in-k

Gusset Geometry

$L = 29.84$ in
 $LA = 25.23$ in
 $LB = 21.31$ in
 $Lg = 18.31$ in

Sway Left

$1.14 F_{cre} A_g = 41.9$ k
 $H_{uc} = 10.7$ k
 $V_{uc} = 14.5$ k
 $M_{uc} = 16.5$ in-k
 $H_{ub} = 19.9$ k
 $V_{ub} = 14.2$ k
 $M_{ub} = 0.0$ in-k
 $0.3 F_{cre} A_g = 12.6$ k
 $H_{uc} = 3.2$ k
 $V_{uc} = 4.4$ k
 $M_{uc} = 4.9$ in-k
 $H_{ub} = 6.0$ k
 $V_{ub} = 4.2$ k
 $M_{ub} = 0.0$ in-k

Lower Gusset

Gusset Edge Forces

Sway Right

$1.14 F_{cre} A_g = 200.9$ k
 $H_{uc} = 50.0$ k
 $V_{uc} = 71.4$ k
 $M_{uc} = 78.1$ in-k
 $H_{ub} = 96.6$ k
 $V_{ub} = 66.0$ k
 $M_{ub} = 0.0$ in-k
 $0.3 F_{cre} A_g = 60.3$ k
 $H_{uc} = 15.0$ k
 $V_{uc} = 21.4$ k
 $M_{uc} = 23.4$ in-k
 $H_{ub} = 29.0$ k
 $V_{ub} = 19.8$ k
 $M_{ub} = 0.0$ in-k

Sway Left

$R_y F_y A_g = 401.7$ k
 $H_{uc} = 99.9$ k
 $V_{uc} = 142.8$ k
 $M_{uc} = 156.1$ in-k
 $H_{ub} = 193.2$ k
 $V_{ub} = 132.0$ k
 $M_{ub} = 0.0$ in-k

Gusset Geometry

$L = 30.64$ in
 $LA = 26.33$ in
 $LB = 22.39$ in
 $Lg = 19.11$ in

Limit States

<u>Lower Brace</u>	<u>Spec Eqn</u>	<u>R_n</u>	<u>φR_n</u>	<u>R_n/φR_n</u>
Brace Weld	J2-4	401.70	501.16	0.80 <i>OK</i>
Net Sect. Rupt.	J4-2	401.70	551.00	0.73 <i>OK</i>
Shear Yield	J4-3	401.70	979.99	0.41 <i>OK</i>
Shear Rupture	J4-4	401.70	841.29	0.48 <i>OK</i>
Reinf. Weld	J2-4	165.00	167.05	0.99 <i>OK</i>
<u>Lower Gusset</u>				
Tension Yield	J4-1	401.70	1160.31	0.35 <i>OK</i>
Buckling	E3	221.00	1033.83	0.21 <i>OK</i>
Block Shear	J4-3	401.70	1053.75	0.38 <i>OK</i>
<u>Lower Gusset at Col.</u>				
Column Weld	J2-4	240.30	383.79	0.63 <i>OK</i>
Tension Yield	J4-1	128.73	973.80	0.13 <i>OK</i>
Shear Yield	J4-2	142.78	649.20	0.22 <i>OK</i>
<u>Lower Gusset at Beam</u>				
Beam Weld	J2-4	292.43	431.49	0.68 <i>OK</i>
Tension Yield	J4-1	131.98	1150.95	0.11 <i>OK</i>
Shear Yield	J4-2	193.16	767.30	0.25 <i>OK</i>
LWY	J10-2	131.98	707.89	0.19 <i>OK</i>
WC	J10-4	131.98	675.16	0.20 <i>OK</i>
<u>Upper Brace</u>				
Brace Weld	J2-4	228.80	501.16	0.46 <i>OK</i>
Net Sect. Rupt.	J4-2	228.80	335.23	0.68 <i>OK</i>
Shear Yield	J4-3	228.80	817.13	0.28 <i>OK</i>
Shear Rupture	J4-4	228.80	701.47	0.33 <i>OK</i>
Reinf. Weld	J2-4	103.13	167.05	0.62 <i>OK</i>
<u>Upper Gusset</u>				
Tension Yield	J4-1	228.80	1092.81	0.21 <i>OK</i>
Buckling	E3	46.12	982.94	0.05 <i>OK</i>
Block Shear	J4-3	228.80	980.63	0.23 <i>OK</i>
<u>Upper Gusset at Col.</u>				
Column Weld	J2-4	137.21	368.60	0.37 <i>OK</i>
Tension Yield	J4-1	75.94	925.30	0.08 <i>OK</i>
Shear Yield	J4-2	79.26	616.87	0.13 <i>OK</i>
<u>Upper Gusset at Beam</u>				
Beam Weld	J2-4	166.44	416.07	0.40 <i>OK</i>
Tension Yield	J4-1	77.23	1101.61	0.07 <i>OK</i>
Shear Yield	J4-2	108.46	734.40	0.15 <i>OK</i>
LWY	J10-2	77.23	680.75	0.11 <i>OK</i>
WC	J10-4	77.23	655.92	0.12 <i>OK</i>
<u>Beam-Column Connection</u>				
BM-Col Weld	J2-4	142.78	417.63	0.34 <i>OK</i>

Limit States (Cont.)

<u>Beam Splice</u>	<u>Spec Eqn</u>	<u>R_u</u>	<u>ϕR_n</u>	<u>$R_u / \phi R_n$</u>	
Bolt Shear (BM)	J3-6	27.50	635.59	0.04 <i>OK</i>	(Gravity Only)
		277.56	763.85	0.36 <i>OK</i>	(Gravity + Seismic)
Bolt Shear (Stub)	J3-6	27.50	707.60	0.04 <i>OK</i>	(Gravity Only)
		277.56	851.83	0.33 <i>OK</i>	(Gravity + Seismic)
Bolt BRG (BM)	J3-10	27.50	459.72	0.06 <i>OK</i>	(Gravity Only)
		277.56	552.50	0.50 <i>OK</i>	(Gravity + Seismic)
Bolt Tear (BM)	J3-10	27.50	337.61	0.08 <i>OK</i>	(Gravity Only)
		277.56	507.17	0.55 <i>OK</i>	(Gravity + Seismic)
Bolt BRG (Stub)	J3-10	27.50	511.81	0.05 <i>OK</i>	(Gravity Only)
		277.56	616.13	0.45 <i>OK</i>	(Gravity + Seismic)
Bolt Tear (Stub)	J3-10	27.50	375.86	0.07 <i>OK</i>	(Gravity Only)
		277.56	565.59	0.49 <i>OK</i>	(Gravity + Seismic)
PL Buckling	E3	276.19	674.45	0.41 <i>OK</i>	
Shear Yield	J4-3	27.50	487.50	0.06 <i>OK</i>	
Flexural Yield	F11-1	820.23	2376.56	0.35 <i>OK</i>	
Shear Rupture	J4-4	27.50	310.78	0.09 <i>OK</i>	
Flexural Rupture	F9-4	820.23	1751.95	0.47 <i>OK</i>	
Ductility	10-3	1.25	1.88	0.66 <i>OK</i>	
Rotation	F2.6b	0.23	1.00	0.23 <i>OK</i>	

Bay Size

$L = 14.67$ ft
 $H_{lower} = 13.75$ ft
 $\theta_{lower} = 46.84$ deg.
 $H_{upper,R} = 13.75$ ft
 $\theta_{upper,R} = 43.16$ deg.
 $H_{upper,L} = 14.58$ ft
 $\theta_{upper,L} = 44.84$ deg.

Members

Beam = W21X93
 Slab Thk. = 5.5 in

Mat'l:

Gusset PL = A36
 Brace = A500-B SQ & RECT
 Reinfr. PL = A572-50
 Beam = A992
 Column = A500-B SQ & RECT

Doubler PL Details:

$t_{PL} = 0.3125$ in
 $h = 8$ in

Upper Brace

HSS5-1/2X5-1/2X3/8

Gusset Details:

$\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 1$ in.
 $t_g = 0.75$ in.
 $L_w = 20$ in.
 Pull-off (R) = 32 in.
 Pull-off (L) = 32 in.
 Clip = 1 in.
 Over Slot = 2 in.
 $D_{brace} = 5$
 $D_{beam} = 5$

Brace Reinforcing Details:

$t_{PL} = 0.5$ in.
 $b = 5$ in.
 $L_w = 9$ in.
 $D_{brace} = 6$

$L = 0$ in
 $D_{brace} = 4$

Lower Brace

HSS5-1/2X5-1/2X3/8

Gusset Details:

$\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 1$ in.
 $t_g = 0.75$ in.
 $L_w = 20$ in.
 Pull-off = 32 in.
 Clip = 1 in.
 Over Slot = 2 in.
 $D_{brace} = 5$
 $D_{beam} = 5$

Brace Reinforcing Details:

$t_{PL} = 0.5$ in.
 $b = 5$ in.
 $L_w = 9$ in.
 $D_{brace} = 6$

$L_{min} = -20.459$ in

Analysis

Welds $F_{EXX} = 70$ ksi
 Weld Ductility Factor = 1.25
 Gusset K Factor = 0.65
 Drag DF = 0.5
 Doubler PL OK? Yes

Status

DESIGN OK!

Upper Gusset

Gusset Edge Forces

Left Brace:

$R_v F_v A_g = 443.072$ k
 $1.14 F_{cre} A_g = 248.1949$ k
 $0.3 F_{cre} A_g = 74.45848$ k
 Right Brace:
 $R_v F_v A_g = 443.072$ k
 $1.14 F_{cre} A_g = 262.8884$ k
 $0.3 F_{cre} A_g = 78.86651$ k
 $N_u = 128.0459$ k
 $V_u = 499.212$ k
 $M_u = 5391.49$ in-k
 $N_{u,eq} = 437.9191$ k
 $N_u = 250.5512$ k
 $V_u = 376.0178$ k
 $M_u = 4060.993$ in-k
 $N_{u,eq} = 483.9548$ k

Gusset Geometry

$L_{Left} = 32.51$ in
 $L_{Right} = 33.03$ in
 $LA = 69.60$ in
 $LB = 20.67$ in
 $Lg, max = 18.74$ in

Lower Gusset

Gusset Edge Forces

$R_v F_v A_g = 443.072$ k
 $1.14 F_{cre} A_g = 262.8884$ k
 $0.3 F_{cre} A_g = 78.86651$ k
 $N_u = 123.2424$ k
 $V_u = 514.997$ k
 $M_u = 5561.967$ in-k
 $N_{u,eq} = 439.7177$ k
 $N_u = 249.1102$ k
 $V_u = 380.7533$ k
 $M_u = 4112.136$ in-k
 $N_{u,eq} = 483.0902$ k

Gusset Geometry

$L = 33.03$ in
 $LA = 70.30$ in
 $LB = 20.53$ in
 $Lg = 18.74$ in

Limit States

	<u>Spec</u>				
<u>Upper Brace</u>	<u>Eqn</u>	<u>R_n</u>	<u>φR_n</u>	<u>R_n/φR_n</u>	
Brace Weld	J2-4	443.1	556.8	0.80	OK
Net Sect. Rupt.	J4-2	443.1	515.4	0.86	OK
Shear Yield	J4-3	443.1	1078.8	0.41	OK
Shear Rupture	J4-4	443.1	874.5	0.51	OK
Reinf. Weld	J2-4	137.5	150.3	0.91	OK
<u>Upper Brace-Gusset</u>					
Tension Yield	J4-1	443.1	694.8	0.64	OK
Buckling	E3	289.2	588.2	0.49	OK
Block Shear	J4-5	443.1	665.4	0.67	OK
<u>Upper Gusset-Beam</u>					
Shear Yield	J4-3	499.2	1127.5	0.44	OK
Tension Yield	J4-1	437.9	1691.2	0.26	OK
		484.0	1691.2	0.29	OK
Flexural Yield	F11-1	5391.5	29424.9	0.18	OK
Interaction		0.23	1.0	0.23	OK
		0.17	1.0	0.17	OK
Beam Weld	J2-4	830.1	1228.3	0.68	OK
		766.1	1308.8	0.59	OK
WLY	J10-2	437.9	2225.6	0.20	OK
		484.0	2225.6	0.22	OK
WC	J10-4	437.9	1772.9	0.25	OK
		484.0	1772.9	0.27	OK
<u>Lower Brace</u>					
Brace Weld	J2-4	443.1	556.8	0.80	OK
Net Sect. Rupt.	J4-2	443.1	515.4	0.86	OK
Shear Yield	J4-3	443.1	1078.8	0.41	OK
Shear Rupture	J4-4	443.1	874.5	0.51	OK
Reinf. Weld	J2-4	137.5	150.3	0.91	OK
<u>Lower Brace-Gusset</u>					
Tension Yield	J4-1	443.1	694.8	0.64	OK
Buckling	E3	289.2	588.2	0.49	OK
Block Shear	J4-5	443.1	665.4	0.67	OK
<u>Lower Gusset-Beam</u>					
Shear Yield	J4-3	515.0	1138.8	0.45	OK
Tension Yield	J4-1	439.7	1708.3	0.26	OK
		483.1	1708.3	0.28	OK
Flexural Yield	F11-1	5562.0	30022.3	0.19	OK
Interaction		0.23	1.0	0.23	OK
		0.17	1.0	0.17	OK
Beam Weld	J2-4	846.5	1234.7	0.69	OK
		768.9	1319.2	0.58	OK
WLY	J10-2	439.7	2246.0	0.20	OK
		483.1	2246.0	0.22	OK
WC	J10-4	439.7	1787.7	0.25	OK
		483.1	1787.7	0.27	OK
<u>Beam</u>					
Shear Yield	G2-1	315.6	450.8	0.70	OK
		234.4	450.8	0.52	OK
Doubler Weld	J2-4	7.52	8.35	0.90	OK

END

2018 IBC Risk Category III Seismic Design Category D

This section presents representative structural calculations for Building 3 (3-story education facility in St. Louis, Missouri) Design Case E. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v3A.

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DESIGN CRITERIA

Search Information

Address: St. Louis, MO, USA

Coordinates: 38.62700249999999, -90.19940419999999

Elevation: 462 ft

Timestamp: 2020-10-07T21:45:59.452Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: III

Site Class: D

MCE_R Horizontal Response Spectrum

Design Horizontal Response Spectrum

Basic Parameters

Name	Value	Description
S _S	0.453	MCE _R ground motion (period=0.2s)
S ₁	0.16	MCE _R ground motion (period=1.0s)
S _{MS}	0.651	Site-modified spectral acceleration value
S _{M1}	0.365	Site-modified spectral acceleration value
S _{DS}	0.434	Numeric seismic design value at 0.2s SA
S _{D1}	0.244	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F _a	1.438	Site amplification factor at 0.2s
F _v	2.279	Site amplification factor at 1.0s
CR _S	0.909	Coefficient of risk (0.2s)
CR ₁	0.863	Coefficient of risk (1.0s)
PGA _G	0.261	MCE _G peak ground acceleration
F _{PGA}	1.339	Site amplification factor at PGA
PGA _M	0.349	Site modified peak ground acceleration
T _L	12	Long-period transition period (s)
SsRT	0.453	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.498	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.16	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.186	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGA _d	0.5	Factored deterministic acceleration value (PGA)

Disclaimer

Hazard loads are provided by the U.S. Geological Survey [Seismic Design Web Services](#).

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Search Information

Address: St. Louis, MO, USA
Coordinates: 38.62700249999999, -90.19940419999999
Elevation: 462 ft
Timestamp: 2020-08-24T21:13:20.719Z
Hazard Type: Wind

ASCE 7-16

MRI 10-Year 74 mph
MRI 25-Year 81 mph
MRI 50-Year 86 mph
MRI 100-Year 93 mph
Risk Category I 101 mph
Risk Category II 106 mph
Risk Category III 114 mph
Risk Category IV 118 mph

ASCE 7-10

MRI 10-Year 76 mph
MRI 25-Year 84 mph
MRI 50-Year 90 mph
MRI 100-Year 96 mph
Risk Category I 105 mph
Risk Category II 115 mph
Risk Category III-IV 120 mph

ASCE 7-05

ASCE 7-05 Wind Speed 90 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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Search Information

Address: St. Louis, MO, USA
Coordinates: 38.62700249999999, -90.19940419999999
Elevation: 462 ft
Timestamp: 2020-08-24T21:23:22.173Z
Hazard Type: Snow

ASCE 7-16

Ground Snow Load 20 lb/sqft

ASCE 7-10

Ground Snow Load 20 lb/sqft

ASCE 7-05

Ground Snow Load 20 lb/sqft

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer.

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DESIGN CRITERIA

VERTICAL LOADS

AREA	DESIGN DEAD LOAD	LIVE LOAD (1)	CONCENTRATED LOADS
CLASSROOM AND OFFICE	75 PSF	50 PSF	1000#
CORRIDORS ABOVE 1ST FLOOR	75 PSF	80 PSF	1000#
ROOF	20 PSF	20 PSF	-

(1) LIVE LOADS EXCEPT SNOW LOADS ARE REDUCED PER IBC SECTION 1607.11.

SNOW:

$P_g = 20$ PSF = GROUND SNOW LOAD

$P_f = 0.7 C_e C_t I_s P_g$ = FLAT ROOF SNOW LOAD

$I_s = 1.1$ $C_e = 1.0$, $C_t = 1.0$. C_s = VARIES

VIBRATION DESIGN CRITERIA:

VIBRATION CONSIDERATIONS WERE DESIGNED PER THE RECOMMENDATIONS IN THE GUIDELINES FOR DESIGN AND CONSTRUCTION OF COMPUTER EQUIPMENT RESIDENCES. FOR CLASSROOM FLOORS A VIBRATION VELOCITY LIMIT OF 8000 MICRO INCHES PER SECOND WAS EVALUATED AT A WALKING SPEED OF 75 STEPS PER MINUTE.

LATERAL FORCES

LATERAL FORCES ARE TRANSMITTED BY DIAPHRAGM ACTION OF ROOF AND FLOORS TO BRACED FRAME/MOMENT FRAME. LOADS ARE THEN TRANSFERRED TO FOUNDATION BY BRACED FRAME/MOMENT FRAME ACTION WHERE ULTIMATE DISPLACEMENT IS RESISTED BY PASSIVE PRESSURE OF EARTH AND/OR SLIDING FRICTION. OVERTURNING IS RESISTED BY DEAD LOAD OF THE STRUCTURE.

WIND

THE BUILDING MEETS THE CRITERIA TO USE THE "ENCLOSED, PARTIALLY ENCLOSED, AND OPEN BUILDING OF ALL HEIGHTS PROCEDURE" PER ASCE 7-16.

- EXPOSURE CATEGORY = B
- BASIC WIND SPEED, (3 SEC. GUST), $V_{ULT} = 114$ MPH; $V_{ASD} = 88$ MPH
- RISK CATEGORY PER IBC TABLE 1604.5 = III
- TOPOGRAPHIC FACTOR $K_{ZT} = 1.0$
- INTERNAL PRESSURE COEFFICIENT (ENCLOSED) = ± 0.18
- BASE SHEAR $V = 146$ KIPS AT BRACE FRAMES, 64 KIPS AT MOMENT FRAME

SEISMIC: (ASCE 7-16) $V = C_s W$

WHERE $C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$; WITH

C_s MINIMUM = 0.044 $S_{DS} I_e \geq 0.01$

OR

C_s MINIMUM = $\frac{0.5 S_1}{\frac{R}{I_e}}$ FOR $S_1 > 0.6g$

C_s MAXIMUM = $\frac{S_{D1}}{T \left(\frac{R}{I_e}\right)}$ FOR $T \leq T_L$

OR

C_s MAXIMUM = $\frac{S_{D1} T_L}{T^2 \left(\frac{R}{I_e}\right)}$ FOR $T > T_L$

SEISMIC IMPORTANCE FACTOR, $I_e = 1.25$

RISK CATEGORY OF BUILDING PER IBC TABLE 1604.5 = III

SPECTRAL RESPONSE ACCELERATIONS $S_s = 0.453$ & $S_1 = 0.16$

SITE CLASS PER TABLE 20.3-1 = D

DESIGN SPECTRAL RESPONSE ACCELERATIONS $S_{DS} = 0.434$ & $S_{D1} = 0.243$

SEISMIC DESIGN CATEGORY = D

W = EFFECTIVE SEISMIC WEIGHT OF BUILDING = 2140 KIPS

ANALYSIS PROCEDURE USED = MODAL RESPONSE SPECTRUM ANALYSIS

RESPONSE MODIFICATION FACTOR PER TABLE 12.2-1, $R = 8.0$ AT SPECIAL STEEL MOMENT FRAMES, $R = 6.0$ AT SPECIAL CONCENTRICALLY BRACED FRAMES

$C_s = 0.047$ AT MOMENT FRAMES, $C_s = 0.09$ AT BRACED FRAMES

DESIGN BASE SHEAR $V = 188.4$ KIPS AT BRACED FRAMES, 97.62 KIPS AT MOMENT FRAMES

FOUNDATION DESIGN CRITERIA

SOIL BEARING PRESSURE: 4000 PSF (ASSUMED)*

ACTIVE PRESSURE - RESTRAINED: 50 PCF +14H SEISMIC SURCHARGE (ASSUMED)

ACTIVE PRESSURE - UNRESTRAINED: 35 PCF +6H SEISMIC SURCHARGE (ASSUMED)

PASSIVE RESISTANCE: 200 PCF (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

COEFFICIENT OF FRICTION: .35 (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

*1/3 INCREASE ALLOWED FOR SEISMIC OR WIND LOADING

ITEM	DESIGN f'_c (PSI) (AT 28 DAYS U.N.O.)	MAX. W/C RATIO	MIN. FLYASH OR SLAG (PCY)	AGGREGATE GRADING ASTM AASHTO	NOTES
SLABS ON GRADE - UNO	4000	0.45	100	57 OR 67	1
FOUNDATIONS - UNO	3000	0.50	--	57 OR 67	
STEM WALLS AND OTHER WALLS - UNO	4000	0.50	100	57 OR 67	
ELEVATED DECKS, TOPPING SLABS, AND SLABS ON METAL	5000	0.40	100	57 OR 67	

GRAVITY DESIGN

NOTE: BENTLEY RAM STRUCTURAL WAS USED AS DESIGN AIDS

ROOF BEAM DESIGN

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Floor Map

02/18/21 12:47:46

Steel Code: AISC 360-10 LRFD

Floor Type: Roof

Beam Numbers

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Beam Design

02/18/21 12:47:46

Steel Code: AISC 360-10 LRFD

Floor Type: Roof**Beam Number = 138****SPAN INFORMATION (ft): I-End (68.75,2.67) J-End (68.75,32.00)**

Beam Size (Optimum) = W16X26

Fy = 50.0 ksi

Total Beam Length (ft) = 29.33

Mp (kip-ft) = 184.17

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.190	0.000	---	Snow	0.000
	29.333	0.190	0.000			0.000
2	0.000	0.026	0.000	---	NonR	0.000
	29.333	0.026	0.000			0.000

SHEAR (Ultimate): Max Vu (1.4DL) = 4.44 kips 0.90Vn = 105.97 kips**MOMENTS (Ultimate):**

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.4DL	32.5	14.7	0.0	1.00	0.90	165.75
Controlling		1.4DL	32.5	14.7	0.0	1.00	0.90	165.75

REACTIONS (kips):

	Left	Right
DL reaction	3.17	3.17
Max +total reaction (factored)	4.44	4.44

DEFLECTIONS:

					Ratio
Dead load (in)	at 14.67 ft =	-0.412	L/D = 853 > 240		0.28
Live load (in)	at 14.67 ft =	-0.000			
Net Total load (in)	at 14.67 ft =	-0.412	L/D = 853 > 240		0.28

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FLOOR BEAM DESIGN

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Floor Map

11/18/20 09:27:36

Steel Code: AISC 360-10 LRFD

Floor Type: LEVEL 2

Beam Numbers

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Beam Design

02/11/21 15:54:03

Steel Code: AISC 360-10 LRFD

Floor Type: LEVEL 2**Beam Number = 135****SPAN INFORMATION (ft): I-End (59.25,2.67) J-End (59.25,32.00)**

Beam Size (Optimum) = W16X26

Fy = 50.0 ksi

Total Beam Length (ft) = 29.33

COMPOSITE PROPERTIES (Not Shored):

		Left	Right
Deck Label		3W+2.5	3W+2.5
Concrete thickness (in)		2.50	2.50
Unit weight concrete (pcf)		150.00	150.00
f _c (ksi)		4.00	4.00
Decking Orientation		perpendicular	perpendicular
Decking type		ASC 3W	ASC 3W
beff (in)	=	88.00	Y bar(in) = 17.42
Mnf (kip-ft)	=	406.66	Mn (kip-ft) = 280.02
C (kips)	=	103.38	PNA (in) = 11.99
I _{eff} (in ⁴)	=	770.41	I _{tr} (in ⁴) = 1205.71
Stud length (in)	=	4.50	Stud diam (in) = 0.75
Stud Capacity (kips)	Q _n = 17.2	R _g = 1.00	R _p = 0.60
# of studs:	Full = 52	Partial = 12	Actual = 12
Number of Stud Rows = 1	Percent of Full Composite Action = 26.92		

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.731	0.487	0.487	12.3%	Red	0.146	0.195
	29.333	0.731	0.487	0.487			0.146	0.195
2	0.000	0.026	0.026	0.000	---	NonR	0.000	0.000
	29.333	0.026	0.026	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 26.80 kips 0.90Vn = 105.97 kips**MOMENTS (Ultimate):**

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	99.8	14.7	0.0	1.00	0.90	165.75
	Init DL	1.4DL	77.3	14.7	---	---		
	Max +	1.2DL+1.6LL	196.5	14.7	---	---	0.90	252.02
Controlling		1.2DL+1.6LL	196.5	14.7	---	---	0.90	252.02

REACTIONS (kips):

	Left	Right
Initial reaction	10.39	10.39
DL reaction	11.11	11.11
Max +LL reaction	8.42	8.42
Max +total reaction (factored)	26.80	26.80

DEFLECTIONS: (Camber = 3/4)

			Ratio
Initial load (in)	at 14.67 ft =	-0.980	L/D = 359 > 200 0.56
Live load (in)	at 14.67 ft =	-0.428	L/D = 823 > 360 0.44

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RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Beam Design

Page 2/2

02/11/21 15:54:03

Steel Code: AISC 360-10 LRFD

Post Comp load (in)	at 14.67 ft = -0.610	L/D = 577 > 240	0.42
Net Total load (in)	at 14.67 ft = -0.840	L/D = 419 > 240	0.57

VIBRATION TO A TYPICAL BAY

FloorVibe V3.00, (C)2006-2018 by Structural Engineers, Inc.
www.floorvibe.com
Licensee: PCS Structural Solutions, Tacoma, Washington

SHEET OF
DATE: 6/25/2020
PAGE: 1 BY: HL

Project ID: ATC-NIST Bldg Comparison
Project # : 19051
Bay ID : Typ Bay

VIBRATION ANALYSIS:

Activity: Walking
Occupancy Category: Electronic Office
Evaluation Criterion: Walking
References: Murray, T.M., Allen, D.E., Ungar, E.E, Davis, D.B.
"Vibrations of Steel-Framed Structural Systems
Due To Human Activity", AISC Design Guide #11 2nd Ed, 2016
Murray, T.M., and Davis, B.,
"Vibration of Steel Joist-Concrete Floor Systems",
SJI Technical Digest No. 5, 2014
Constant Force, $P_o = 65.1b$
Modal Damping Ratio, $\beta = 0.025$
Acceleration Limit, $a_o/g \times 100\% = 0.50 \%$
Beams are not continuous
Girders are not continuous at columns

EVALUATION: Combined mode $a_p/g = 0.514 \%$ > 0.50%
This peak acceleration MAY BE ACCEPTABLE,
ENGINEERING JUDGEMENT IS REQUIRED.

PARAMETER SUMMARY:

Parameter	Beam Panel	Left Girder Panel	Right Girder Panel	System
Member	W16X40	W18X50	W18X50	
Uniform Load, plf	663.3	1307.0	1003.5	
Effective I, in ⁴	1924.0	2790.3	2507.2	
Frequency, Hz	8.27	6.52	7.05	5.12
Panel Width, ft	26.75	50.67	50.67	
Panel Weight, k	51.0	104.8	106.1	84.2

FRAMING:

Girder Span = 30.00 ft
Beam Spans:
Left = 9.15 ft
Center = 28.75 ft
Right = 0.00 ft
Girders/Walls:
Left -W18X50
Right-W18X50
Beam -W16X40
3 spaces at 120.00 in
Floor Width =120.00 ft
Floor Length= 76.00 ft
Concrete: dc = 5.50 in
f'c = 3.00 ksi
wt. = 145.0 pcf
Deck Height = 3.00 in

Loading: Dead = 4.00 psf
Live = 8.00 psf
Collateral = 0.00 psf

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The recommended limit for offices and residences is 0.5%g. Between 0.5%g and 0.55%g, the shaded area, there are known cases that have not resulted in complaints and engineering judgement is required in this range.

Note: Floors with a frequency below 3 Hz are not recommended

COLUMN DESIGN

Floor Map

11/18/20 09:27:36

Steel Code: AISC 360-10 LRFD

Floor Type: LEVEL 2

Beam Numbers

RAM Structural System

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

Gravity Column Design

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02/11/21 16:03:27

Steel Code: AISC 360-10 LRFD

Story level Level 2, Column Line 6-C, Column # 31

Fy (ksi) = 50.00

Column Size

= W10X33

Orientation (deg.) = 90.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	13.75	13.75
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	7.36	6.48
Bottom _____	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	72.01	31.85	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 137.37	0.90Pnx (kip) = 390.25	Pu/0.90Pnx = 0.352
	0.90Pny (kip) = 257.68	Pu/0.90Pny = 0.533
	0.90Pn (kip) = 257.68	Pu/0.90Pn = 0.533

DEMAND/CAPACITY LIMIT FOR STRENGTH : 1.000**CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 10:**

	Dead	Live	Roof
Axial (kip) _____	72.01	28.24	0.00
Moments Top Mx (kip-ft) _____	-2.32	-1.20	0.00
My (kip-ft) _____	-0.61	-1.28	0.00
Bot Mx (kip-ft) _____	0.00	0.00	0.00
My (kip-ft) _____	0.00	0.00	0.00

Single curvature about X-Axis

Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 131.59	0.90*Pn (kip) = 257.68
Mux (kip-ft) = -4.71	0.90*Mnx (kip-ft) = 145.50
Muy (kip-ft) = -2.78	0.90*Mny (kip-ft) = 52.50
Rm = 1.00	
Cbx = 1.67	
Cmx = 0.60	Cmy = 0.60
Pex (kip) = 1797.74	Pey (kip) = 384.78
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.511

RAM Structural System

Gravity Column Design

RAM Steel 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 RC 2 hl

Building Code: IBC

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Steel Code: AISC 360-10 LRFD

$$\text{Eq H1-1a: } 0.511 + 0.029 + 0.047 = 0.587$$

LATERAL DESIGN

SEISMIC FORCE

IBC 2018 RISK Category III Education Building

From Spreadsheet,

N-S Direction, SCBF used as force resist element, R=6, Omega=2. Cs=0.09

E-W Direction, SMF used as force resist element, R=8, Omega=3, Cs=CsMax=0.067

N-S:

$$I_{ns} := 1.25$$

$$R_{ns} := 6$$

From Table 12.2-1

$$\Omega_{ns} := 2.0$$

$$Cd_{ns} := 5$$

$$h_1 := 13.75$$

$$\text{Height1} := 13.75 \quad \text{ft}$$

$$h_2 := 27.5 - 13.75 = 13.75$$

$$\text{Height2} := 13.75 \quad \text{ft}$$

$$h_3 := 14.5$$

$$\text{Height3} := 14.5 \quad \text{ft}$$

Allowable Story Drift: Δa_{ns}

$$\Delta a_{ns} := 0.020 \cdot h_{sx}$$

from Table 12.12-1

Max deflection(Drift):

from 12.8-15

Roof:

$$h_{sx} := h_1 \cdot 12$$

$$\Delta a_{ns} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ns} = 3.3$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ns}}{Cd_{ns}} = 0.825 \quad \text{in}$$

Third Floor:

$$h_{sx} := h_2 \cdot 12$$

$$\Delta a_{ns} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ns} = 3.3$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ns}}{Cd_{ns}} = 0.825 \quad \text{in}$$

Second Floor:

$$h_{sx} := h_3 \cdot 12$$

$$\Delta a_{ns} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ns} = 3.48$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ns}}{Cd_{ns}} = 0.87 \quad \text{in}$$

E-W:

From Table 12.2-1

$$I_{ew} := 1.25$$

$$R_{ew} := 8$$

$$\Omega_{ew} := 3$$

$$Cd_{ew} := 5.5$$

From Table 12.2-1

$$h_1 := 13.75 \quad \text{ft}$$

$$h_2 := 27.5 - 13.75 = 13.75 \quad \text{ft}$$

$$h_3 := 14.5 \quad \text{ft}$$

Allowable Story Drift: Δa_{ns}

$$\Delta a_{ew} := 0.02 \cdot h_{sx}$$

from Table 12.12-1

Max deflection(Drift):

from 12.8-15

Roof:

$$h_{sx} := h_1 \cdot 12$$

$$\Delta a_{ew} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ew} = 3.3$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ew}}{Cd_{ew}} = 0.75 \quad \text{in}$$

Third Floor:

$$h_{sx} := h_2 \cdot 12$$

$$\Delta a_{ew} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ew} = 3.3$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ew}}{Cd_{ew}} = 0.75 \quad \text{in}$$

Second Floor:

$$h_{sx} := h_3 \cdot 12$$

$$\Delta a_{ew} := 0.02 \cdot h_{sx}$$

$$\delta_x := \Delta a_{ew} = 3.48$$

$$\delta_{xe} := \delta_x \cdot \frac{I_{ew}}{Cd_{ew}} = 0.791 \quad \text{in}$$

Check Seismic Weight:

$$DL_w := 20 \text{ psf}$$

Diaphragm 1:

Floor Dead Load	$DL_1 := 80 \text{ psf}$	(Partional load = 10psf included as Seismi Weight)
Floor Area	$FloorA_1 := 9860 \text{ sf}$	
Building Perimeter	$P_1 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$	
Wal Height	$H_1 := \frac{(Height2 + Height1)}{2} = 13.75 \text{ ft}$	
	$W_1 := (DL_1 \cdot FloorA_1 + P_1 \cdot H_1 \cdot DL_w) = 9.267 \times 10^5 \text{ lb}$	

Diaphragm 2:

Floor Dead Load	$DL_2 := 80 \text{ psf}$	(Partional load = 10psf included as Seismi Weight)
Floor Area	$FloorA_2 := 10100 \text{ sf}$	
Building Perimeter	$P_2 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$	
Wal Height	$H_2 := \frac{(Height2 + Height3)}{2} = 14.125 \text{ ft}$	
	$W_2 := DL_2 \cdot FloorA_2 + P_2 \cdot H_2 \cdot DL_w = 9.497 \times 10^5 \text{ lb}$	

Diaphragm 3(ROOF):

Floor Dead Load	$DL_3 := 20 \text{ psf}$	
Floor Area	$FloorA_3 := 9560 \text{ sf}$	
Building Perimeter	$P_3 := (174.5 + 76.3) \cdot 2 = 501.6 \text{ ft}$	
Wal Height	$H_3 := \frac{(Height3)}{2} = 7.25 \text{ ft}$	
	$W_3 := DL_3 \cdot FloorA_3 + P_3 \cdot H_3 \cdot DL_w = 2.639 \times 10^5 \text{ lb}$	
	$\underline{W} := W_1 + W_2 + W_3 = 2.14 \times 10^6 \text{ lb}$	

Check Torsional Irregularity

By inspection, we notice there will be a torsional irregularity of Type 1a ($\delta_{avg} > 1.2(\text{Max}\delta)$) in Y direction
See next page for Torsional Irregularity Calc

$$\frac{\delta_{\max}}{\delta_{\text{avg}}} = 1.347 \quad A_x := \left(\frac{\delta_{\max}}{1.2\delta_{\text{avg}}} \right)^2 = 1.26$$

When $A_x=1.0$ 5% of dimension of structure will be applied on both direction

Now in Y direction:

$$0.05 \cdot A_x = 0.063$$

New accidental moment 0.063% will be apply to design in Y direction

Redundancy Factor p , per 12.3.4.2 permitted to be taken as 1.0 for SDC C building

RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

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LOAD CASE: EQ

Seismic ASCE 7-16 Equivalent Lateral Force

Importance Factor: 1.25 TL: 12.00 s

Site Class D: Stiff Soil, Default

Ss: 0.453 g S1: 0.160 g

Fa: 1.438 Fv: 2.280 SDs: 0.434 g SD1: 0.243 g

Use Specified Seismic Design Category: C

Provisions for: Force

Ground Level: Base

Dir	Eccent	R	Ta Equation				Building Period-T		
X	+ And -	8.0	Std,Ct=0.028,x=0.80				Calculated		
Y	+ And -	6.0	Std,Ct=0.030,x=0.75				Calculated		
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
X	0.559	1.457	1.372	0.814	0.068	0.047	0.024	0.047	1.157
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
Y	0.496	1.457	0.375	0.375	0.090	0.135	0.024	0.090	1.000

Total Building Weight (kips) = 2090.71

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	19.96	0.00	70.05	38.26
Level 3	1	27.50	54.03	0.00	72.11	40.86
Level 2	1	13.75	23.64	0.00	66.05	41.87

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	19.956	0.000
Level 3	1	54.028	0.000
Level 2	1	23.636	0.000
		97.62	0.00

APPLIED STORY FORCES

Type: EQ_ASCE716_X+E_F

Level	Ht ft	Fx kips	Fy kips
-------	----------	------------	------------

RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

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Roof	42.17	19.96	0.00
Level 3	27.50	54.03	0.00
Level 2	13.75	23.64	0.00
		<hr/>	<hr/>
		97.62	0.00

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	19.96	0.00	70.05	31.08
Level 3	1	27.50	54.03	0.00	72.11	33.02
Level 2	1	13.75	23.64	0.00	66.05	34.03

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	19.956	0.000
Level 3	1	54.028	0.000
Level 2	1	23.636	0.000
		<hr/>	<hr/>
		97.62	0.00

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	19.96	0.00
Level 3	27.50	54.03	0.00
Level 2	13.75	23.64	0.00
		<hr/>	<hr/>
		97.62	0.00

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	0.00	35.63	80.09	34.67
Level 3	1	27.50	0.00	103.16	82.15	36.94
Level 2	1	13.75	0.00	50.31	76.09	37.95

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx	Sum Fy
-------	---------	--------	--------

RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

Loads and Applied Forces

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		kip	kip
Roof	1	0.000	35.631
Level 3	1	0.000	103.157
Level 2	1	0.000	50.315
		<hr/>	<hr/>
		0.00	189.10

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Ht ft	Fx kip	Fy kip
Roof	42.17	0.00	35.63
Level 3	27.50	0.00	103.16
Level 2	13.75	0.00	50.31
		<hr/>	<hr/>
		0.00	189.10

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Diaph.#	Ht ft	Fx kip	Fy kip	X ft	Y ft
Roof	1	42.17	0.00	35.63	60.01	34.67
Level 3	1	27.50	0.00	103.16	62.07	36.94
Level 2	1	13.75	0.00	50.31	56.01	37.95

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kip	Sum Fy kip
Roof	1	0.000	35.631
Level 3	1	0.000	103.157
Level 2	1	0.000	50.315
		<hr/>	<hr/>
		0.00	189.10

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Ht ft	Fx kip	Fy kip
Roof	42.17	0.00	35.63
Level 3	27.50	0.00	103.16
Level 2	13.75	0.00	50.31
		<hr/>	<hr/>
		0.00	189.10

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RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

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LOAD MULTIPLIERS FOR ECCENTRIC LOADING ON SEMIRIGID DIAPHRAGM:SEISMIC AND RESPONSE SPECTRA LOAD CASES**Story : Level 3 Diaph. #: 1**

Loading Direction	Top Region	Bottom Region	Left Region	Right Region
+X	1.21	0.80	---	---
-X	0.79	1.20	---	---
+Y	---	---	0.74	1.25
-Y	---	---	1.26	0.75

Story : Level 2 Diaph. #: 1

Loading Direction	Top Region	Bottom Region	Left Region	Right Region
+X	1.20	0.82	---	---
-X	0.80	1.18	---	---
+Y	---	---	0.73	1.27
-Y	---	---	1.27	0.73

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LOAD CASE: Winds

ASCE 7-16

Exposure: B

Basic Wind Speed (mph): 114.0

Apply Directionality Factor, $K_d = 0.85$ Use Topography Factor, $K_{zt} = 1.00$ Ground Elevation Factor, $K_e = 1.00$

Use Calculated Frequency for X-Dir.

Use Calculated Frequency for Y-Dir.

Gust Factor for Flexible Structures, G : Use Calculated G for X-Dir.Gust Factor for Rigid Structures, G : Use Calculated G for Y-Dir.

Damping Ratio for Flexible Structures = 0.01

Mean Roof Height (ft): Top Story Height + Parapet = 42.17

Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 0.729 Structure is Flexible

Y-Direction: Natural Frequency = 2.667 Structure is Rigid

 $C_{p\text{Windward}} = 0.80$ $q_{\text{Leeward}} (q_h) = 21.84 \text{ psf}$ $G_{Cp_n} (\text{Parapet}):$ Windward = 1.50 Leeward = -1.00

Height ft	K_z	K_{zt}	q_z psf	Gust Factor G		$C_{p\text{Leeward}}$		Pressure (psf)	
				X	Y	X	Y	X	Y
42.17	0.772	1.000	21.836	1.008	0.820	-0.289	-0.500	23.980	23.274
27.50	0.683	1.000	19.326	0.997	0.820	-0.298	-0.500	21.900	21.627
13.75	0.575	1.000	16.253	0.997	0.820	-0.298	-0.500	19.449	19.612
0.00	0.575	1.000	16.253	0.997	0.820	-0.298	-0.500	19.449	19.612

Windward and Leeward Wind Pressures for Semirigid Diaphragms:

Story	Diaph. #	Windward Pressure (psf)		Leeward Pressure (psf)	
		X	Y	X	Y
Level 3	1	15.410	12.676	6.490	8.952
Level 2	1	12.959	10.660	6.490	8.952

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_X

Level	Diaph.#	Ht ft	F_x kips	F_y kips	X ft	Y ft
Roof	1	42.17	12.27	0.00	68.75	34.92
Level 3	1	27.50	23.26	0.00	68.75	36.50
Level 2	1	13.75	21.39	0.00	68.75	38.17

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum F_x kips	Sum F_y kips
Roof	1	12.267	0.000

RAM Structural System

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3

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Loads and Applied Forces

Level 3	1	23.256	0.000
Level 2	1	21.388	0.000
		<hr/> 56.91	<hr/> 0.00

APPLIED STORY FORCES

Type: Wind_ASCE716_1_X

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	12.27	0.00
Level 3	27.50	23.26	0.00
Level 2	13.75	21.39	0.00
		<hr/> 56.91	<hr/> 0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	42.17	0.00	26.58	68.75	34.92
Level 3	1	27.50	0.00	48.92	68.75	38.17
Level 2	1	13.75	0.00	43.75	68.75	38.17

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	0.000	26.581
Level 3	1	0.000	55.129
Level 2	1	0.000	47.656
		<hr/> 0.00	<hr/> 129.37

APPLIED STORY FORCES

Type: Wind_ASCE716_1_Y

Level	Ht ft	Fx kips	Fy kips
Roof	42.17	0.00	26.58
Level 3	27.50	0.00	48.92
Level 2	13.75	0.00	43.75
		<hr/> 0.00	<hr/> 119.25

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RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Drift

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Steel Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects

Member Force Output: At Face of Joint

P-Delta: Yes Scale Factor (DL): 1.20 Scale Factor (LL): 1.00
 Scale Factor (Roof): 1.00 Scale Factor (Snow): 1.00

Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
Dyn9	Scaled Drift	Dyn_ASCE716_SRSS_X_+E
Dyn10	Scaled Drift	Dyn_ASCE716_SRSS_X_-E
Dyn11	Scaled Drift	Dyn_ASCE716_SRSS_Y_+E
Dyn12	Scaled Drift	Dyn_ASCE716_SRSS_Y_-E

Displacements for semirigid diaphragm are reported based on nodal displacements near to selected point.

RESULTS:

Location (ft): (118.000, 69.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	-0.0716	-0.0009	-0.0194	-0.0001	0.0001	0.0000
	Lp	-0.0376	0.0053	-0.0090	0.0033	0.0001	0.0000
	Ln	0.0008	0.0001	0.0002	0.0001	0.0000	0.0000
	Dyn9	4.9271	0.0158	0.6554	0.0151	0.0037	0.0001
	Dyn10	4.8451	0.1203	0.6385	0.0378	0.0036	0.0002
	Dyn11	-0.1493	0.7125	-0.0305	0.1699	0.0002	0.0010
	Dyn12	0.4250	0.9477	0.0892	0.2058	0.0005	0.0012
Level 3	D	-0.0522	-0.0008	-0.0247	0.0001	0.0001	0.0000
	Lp	-0.0287	0.0020	-0.0140	0.0024	0.0001	0.0000
	Ln	0.0006	0.0001	0.0003	0.0001	0.0000	0.0000
	Dyn9	4.2937	0.0096	1.3803	0.0120	0.0084	0.0001
	Dyn10	4.2258	0.0868	1.3513	0.0482	0.0082	0.0003
	Dyn11	-0.1303	0.5560	-0.0500	0.2745	0.0003	0.0017
	Dyn12	0.3756	0.7504	0.1506	0.3620	0.0009	0.0022
Level 2	D	-0.0275	-0.0009	-0.0275	-0.0009	0.0002	0.0000
	Lp	-0.0146	-0.0004	-0.0146	-0.0004	0.0001	0.0000
	Ln	0.0004	0.0000	0.0004	0.0000	0.0000	0.0000

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Drift

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Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	Dyn9	2.9216	-0.0125	2.9216	-0.0125	0.0177	0.0001
	Dyn10	2.8817	0.0404	2.8817	0.0404	0.0175	0.0002
	Dyn11	-0.0849	0.2847	-0.0849	0.2847	0.0005	0.0017
	Dyn12	0.2373	0.3926	0.2373	0.3926	0.0014	0.0024

Location (ft): (118.000, 3.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	-0.0828	0.0000	-0.0247	0.0010	0.0001	0.0000
	Lp	-0.0381	0.0056	-0.0086	0.0035	0.0000	0.0000
	Ln	0.0010	0.0001	0.0003	0.0000	0.0000	0.0000
	Dyn9	4.9636	0.0138	0.6946	0.0125	0.0039	0.0001
	Dyn10	5.0500	0.1157	0.7191	0.0345	0.0041	0.0002
	Dyn11	0.1831	0.6953	0.0535	0.1638	0.0003	0.0009
	Dyn12	-0.5611	0.9271	0.1875	0.2021	0.0011	0.0011
Level 3	D	-0.0581	-0.0010	-0.0290	0.0001	0.0002	0.0000
	Lp	-0.0295	0.0021	-0.0142	0.0026	0.0001	0.0000
	Ln	0.0008	0.0001	0.0004	0.0001	0.0000	0.0000
	Dyn9	4.2928	0.0092	1.3954	0.0099	0.0085	0.0001
	Dyn10	4.3582	0.0846	1.4261	0.0456	0.0086	0.0003
	Dyn11	0.1358	0.5414	0.0671	0.2645	0.0004	0.0016
	Dyn12	-0.3918	0.7316	-0.2234	0.3503	0.0014	0.0021
Level 2	D	-0.0290	-0.0011	-0.0290	-0.0011	0.0002	0.0000
	Lp	-0.0153	-0.0005	-0.0153	-0.0005	0.0001	0.0000
	Ln	0.0004	0.0000	0.0004	0.0000	0.0000	0.0000
	Dyn9	2.9052	-0.0109	2.9052	-0.0109	0.0176	0.0001
	Dyn10	2.9407	0.0405	2.9407	0.0405	0.0178	0.0002
	Dyn11	0.0705	0.2796	0.0705	0.2796	0.0004	0.0017
	Dyn12	-0.1851	0.3847	-0.1851	0.3847	0.0011	0.0023

Location (ft): (0.000, 69.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	-0.0702	0.0209	-0.0183	0.0109	0.0001	0.0001
	Lp	-0.0372	0.0064	-0.0089	0.0026	0.0001	0.0000
	Ln	0.0008	-0.0003	0.0002	-0.0001	0.0000	0.0000
	Dyn9	4.9560	-0.0337	0.6801	0.0287	0.0039	0.0002
	Dyn10	4.8696	-0.2020	0.6580	-0.0564	0.0037	0.0003
	Dyn11	-0.1661	1.0500	-0.0348	0.2310	0.0002	0.0013

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Drift

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Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	Dyn12	0.4436	1.2088	0.1068	0.2740	0.0006	0.0016
Level 3	D	-0.0519	0.0100	-0.0240	0.0087	0.0001	0.0001
	Lp	-0.0283	0.0038	-0.0134	0.0032	0.0001	0.0000
	Ln	0.0006	-0.0002	0.0003	-0.0001	0.0000	0.0000
	Dyn9	4.3027	0.0193	1.3834	-0.0196	0.0084	0.0001
	Dyn10	4.2345	-0.1501	1.3550	-0.0832	0.0082	0.0005
	Dyn11	-0.1384	0.8258	-0.0530	0.3704	0.0003	0.0022
	Dyn12	0.3784	0.9500	0.1511	0.4292	0.0009	0.0026
Level 2	D	-0.0279	0.0013	-0.0279	0.0013	0.0002	0.0000
	Lp	-0.0149	0.0007	-0.0149	0.0007	0.0001	0.0000
	Ln	0.0004	-0.0000	0.0004	-0.0000	0.0000	0.0000
	Dyn9	2.9282	0.0278	2.9282	0.0278	0.0177	0.0002
	Dyn10	2.8871	-0.0713	2.8871	-0.0713	0.0175	0.0004
	Dyn11	-0.0886	0.4612	-0.0886	0.4612	0.0005	0.0028
	Dyn12	0.2434	0.5292	0.2434	0.5292	0.0015	0.0032

Location (ft): (0.000, 3.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	-0.0856	0.0203	-0.0272	0.0107	0.0002	0.0001
	Lp	-0.0388	0.0062	-0.0096	0.0028	0.0001	0.0000
	Ln	0.0011	-0.0003	0.0003	-0.0001	0.0000	0.0000
	Dyn9	4.9341	-0.0378	0.6778	-0.0331	0.0039	0.0002
	Dyn10	5.0193	-0.1969	0.6994	-0.0575	0.0040	0.0003
	Dyn11	0.1925	1.0514	0.0489	0.2209	0.0003	0.0013
	Dyn12	-0.5498	1.2090	-0.1689	0.2647	0.0010	0.0015
Level 3	D	-0.0584	0.0097	-0.0296	0.0080	0.0002	0.0000
	Lp	-0.0292	0.0034	-0.0137	0.0025	0.0001	0.0000
	Ln	0.0007	-0.0002	0.0004	-0.0001	0.0000	0.0000
	Dyn9	4.2754	0.0257	1.3902	-0.0226	0.0084	0.0001
	Dyn10	4.3417	-0.1436	1.4199	-0.0815	0.0086	0.0005
	Dyn11	0.1459	0.8377	0.0697	0.3737	0.0004	0.0023
	Dyn12	-0.3967	0.9599	-0.2129	0.4301	0.0013	0.0026
Level 2	D	-0.0288	0.0017	-0.0288	0.0017	0.0002	0.0000
	Lp	-0.0155	0.0010	-0.0155	0.0010	0.0001	0.0000
	Ln	0.0004	-0.0000	0.0004	-0.0000	0.0000	0.0000
	Dyn9	2.8919	0.0350	2.8919	0.0350	0.0175	0.0002
	Dyn10	2.9292	-0.0662	2.9292	-0.0662	0.0178	0.0004
	Dyn11	0.0780	0.4708	0.0780	0.4708	0.0005	0.0029

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Drift

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Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	Dyn12	-0.1987	0.5394	-0.1987	0.5394	0.0012	0.0033

TORSIONAL IRREGULARITY DATA:

X-Axis:

Story	LdC	Drift in	Coord ft	Drift in	Coord ft	Max/Min	Max/Ave
Roof	Dyn10	0.7191	(118.00 3.00)	0.6385	(118.00 69.00)	1.126	1.059
Level 3	Dyn10	1.4261	(118.00 3.00)	1.3513	(118.00 69.00)	1.055	1.027
Level 2	Dyn10	2.9407	(118.00 3.00)	2.8817	(118.00 69.00)	1.020	1.010

Y-Axis:

Story	LdC	Drift in	Coord ft	Drift in	Coord ft	Max/Min	Max/Ave
Roof	Dyn12	0.2021	(118.00 3.00)	0.2740	(0.00 69.00)	1.356	1.151
Level 3	Dyn11	0.3737	(0.00 3.00)	0.2645	(118.00 3.00)	1.413	1.171
Level 2	Dyn11	0.4708	(0.00 3.00)	0.2796	(118.00 3.00)	1.684	1.255

MAX DRIFT < LIMIT

NO EXTREME
TORSIONAL
IRREGULARITY

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RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3 SDC D

Building Code: IBC

04/28/21 15:40:51

Steel Code: AISC360-16 LRFD

CRITERIA:

Rigid End Zones:	Ignore Effects
Member Force Output:	At Face of Joint
P-Delta:	Yes
Scale Factor (DL):	1.20
Scale Factor (LL):	1.00
Scale Factor (Roof):	1.00
Scale Factor (Snow):	1.00

Ground Level: Base

Mesh Criteria :

Max. Distance Between Nodes on Mesh Line (ft) : 4.00

Merge Node Tolerance (in) : 0.0150

Geometry Tolerance (in) : 0.0100

Walls Out-of-plane Stiffness Not Included in Analysis.

Sign considered for Dynamic Load Case Results.

Rigid Links Included at Fixed Beam-to-Wall Locations

Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

LOAD COMBINATION CRITERIA:

Live Load factor f1 (0.5 or 1.0)	0.500
Sds (for Ev)	0.434
RhoX	1.300
RhoY	1.300

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
Dyn5	Scaled MR	Dyn_ASCE716_SRSS_X_+E
Dyn6	Scaled MR	Dyn_ASCE716_SRSS_X_-E
Dyn7	Scaled MR	Dyn_ASCE716_SRSS_Y_+E
Dyn8	Scaled MR	Dyn_ASCE716_SRSS_Y_-E

LOAD COMBINATIONS: IBC 2018 / ASCE 7-16 LRFD

1	*	1.400 D
2	*	1.200 D + 1.600 Lp
3	*	1.200 D + 1.600 Ln
4	*	1.287 D + 0.500 Lp + 1.300 Dyn5
5	*	1.287 D + 0.500 Lp + 1.300 Dyn6
6	*	1.287 D + 0.500 Lp + 1.300 Dyn7
7	*	1.287 D + 0.500 Lp + 1.300 Dyn8
8	*	1.287 D + 0.500 Lp - 1.300 Dyn5
9	*	1.287 D + 0.500 Lp - 1.300 Dyn6
10	*	1.287 D + 0.500 Lp - 1.300 Dyn7
11	*	1.287 D + 0.500 Lp - 1.300 Dyn8
12	*	1.287 D + 0.500 Ln + 1.300 Dyn5
13	*	1.287 D + 0.500 Ln + 1.300 Dyn6
14	*	1.287 D + 0.500 Ln + 1.300 Dyn7
15	*	1.287 D + 0.500 Ln + 1.300 Dyn8
16	*	1.287 D + 0.500 Ln - 1.300 Dyn5

RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3 SDC D

Building Code: IBC

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Steel Code: AISC360-16 LRFD

17	*	1.287 D + 0.500 Ln - 1.300 Dyn6
18	*	1.287 D + 0.500 Ln - 1.300 Dyn7
19	*	1.287 D + 0.500 Ln - 1.300 Dyn8
20	*	1.287 D + 1.300 Dyn5
21	*	1.287 D + 1.300 Dyn6
22	*	1.287 D + 1.300 Dyn7
23	*	1.287 D + 1.300 Dyn8
24	*	1.287 D - 1.300 Dyn5
25	*	1.287 D - 1.300 Dyn6
26	*	1.287 D - 1.300 Dyn7
27	*	1.287 D - 1.300 Dyn8
28	*	0.813 D + 1.300 Dyn5
29	*	0.813 D + 1.300 Dyn6
30	*	0.813 D + 1.300 Dyn7
31	*	0.813 D + 1.300 Dyn8
32	*	0.813 D - 1.300 Dyn5
33	*	0.813 D - 1.300 Dyn6
34	*	0.813 D - 1.300 Dyn7
35	*	0.813 D - 1.300 Dyn8

* = Load combination currently selected to use

Frame #1:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
12	13.40	-3.49	-1.92	-0.46	-0.02	9	0.03 H1-1b	50	W14X68
13	14.30	2.05	-2.18	0.42	-0.01	8	0.03 H1-1b	50	W14X68
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
31	8.98	10.57	0.03	-3.47	0.01	11	0.01 H1-1b	50	W24X131
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
22	15.30	0.00	0.00	-0.00	-0.00	18	0.63 Axial	50	HSS3.5X3.5X3/8
23	17.06	0.00	0.00	0.00	0.00	6	0.70 Axial	50	HSS3.5X3.5X3/8

Level: Level 3

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
13	105.64	0.00	0.00	-0.60	-0.02	10	0.16 Axial	50	W14X68
14	123.56	0.00	0.00	0.67	-0.05	6	0.19 Axial	50	W14X68

RAM Structural System

Code Check Summary

RAM Frame 17.01.01.05

DataBase: GRCC St. Louis ASCE 7-16 hl RC3 SDC D

Building Code: IBC

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Steel Code: AISC360-16 LRFD

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
35	-2.54	175.86	0.00	-23.37	0.01	2	0.32 H1-1b	50	W18X71

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
18	57.71	0.00	0.00	-0.00	-0.00	6	0.49 Axial	50	HSS5.5X5.5X3/8
19	53.65	0.00	0.00	-0.00	-0.00	10	0.45 Axial	50	HSS5.5X5.5X3/8

Level: Level 2

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
13	146.15	3.03	0.68	-0.18	-0.06	11	0.24 H1-1a	50	W14X68
14	175.35	-1.65	0.81	-0.26	-0.07	7	0.28 H1-1a	50	W14X68

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
36	-0.67	-42.03	-0.01	16.59	-0.00	2	0.05 H1-1b	50	W21X93

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
10	96.76	0.00	0.00	0.00	0.00	10	0.82 Axial	50	HSS5.5X5.5X3/8
11	99.06	0.00	0.00	-0.00	0.00	6	0.84 Axial	50	HSS5.5X5.5X3/8

MOMENT FRAME DESIGN

Seismic Provisions Member Code Check Summary

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Frame #2:

Story:Roof **Column: 18** **Type: Special Moment Resisting Frame**

Size:W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Roof **Column: 35** **Type: Special Moment Resisting Frame**

Size:W18X130 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Roof **Column: 36** **Type: Special Moment Resisting Frame**

Size:W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Roof **Beam:126** **Type: Special Moment Resisting Frame**

Size:W14X68 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story:Roof **Beam:127** **Type: Special Moment Resisting Frame**

Size:W14X68 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story:Level 3 **Column: 20** **Type: Special Moment Resisting Frame**

Size:W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story:Level 3 **Column: 44** **Type: Special Moment Resisting Frame**

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Size: W18X130 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 3**Column: 45****Type: Special Moment Resisting Frame**

Size: W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 3**Beam: 140****Type: Special Moment Resisting Frame**

Size: W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story: Level 3**Beam: 141****Type: Special Moment Resisting Frame**

Size: W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- OK

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

E3.6b Beam-to-Column Connection Requirements

Story: Level 2**Column: 26****Type: Special Moment Resisting Frame**

Size: W18X130 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 2**Column: 32****Type: Special Moment Resisting Frame**

Size: W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- OK

E3.5a Basic Requirements --- OK

E3.6g Column Splices

Story: Level 2**Column: 48****Type: Special Moment Resisting Frame**

Size: W18X86 Fy (ksi): 50.00

D1.4a Required Strength --- OK

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RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

D2.5b Column Splices - Required Strength

E3.4c(1) Unbraced Connections --- **OK**

E3.5a Basic Requirements --- **OK**

E3.6g Column Splices

Story:Level 2

Beam: 71

Type: Special Moment Resisting Frame

Size:W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- **OK**

E3.5a Basic Requirements (D1.1 Highly Ductile) --- **OK**

E3.6b Beam-to-Column Connection Requirements

Story:Level 2

Beam:149

Type: Special Moment Resisting Frame

Size:W21X62 Fy (ksi): 50.00

E3.4b Lateral Bracing of Beams (D1.1 Highly Ductile) --- **OK**

E3.5a Basic Requirements (D1.1 Highly Ductile) --- **OK**

E3.6b Beam-to-Column Connection Requirements

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**CHECK BEAM
SAMPLE CALC.**

RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

02/18/21 21:40:05

Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Level 2

Frame No: 2

Member No: 71

Fy (ksi): 50.00

Size: W21X62

Frame Type: Special Moment Resisting Frame

Apply AISC 358 Provisions to Moment Frame

E3.4b Stability Bracing of Beams (D1.2b Highly Ductile) --- OK

Max Lu (in) = 84.00

Lu Limit = 88.79

OK

Lateral Bracing Requirements along Beam

Required strength of lateral brace along beam (A-6-7) = 7.77 kip

Required stiffness of bracing (A-6-8) = 5180.28 kip / Lb

Lateral Bracing Requirements at Plastic Hinge

Required strength of lateral brace at hinge = 23.31 kip

Required stiffness of bracing (A-6-8) = 5180.28 kip / Lb

*Cd = 1.0 assumed for eqns (A-6-7/8)**Lb = distance between braces (in)**The brace closest to the point of inflection should be designed for two times these values (Cd=2)*

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

For calculation of web limit: Pu (kip) = 0.77 ϕP_y (kip) = 823.50

Flange b/tf = 6.70

Limit =

7.35

OK

Web h/tw = 46.90

Limit =

59.01

OK

E3.6b Beam-to-Column Connection Requirements

*Beam-to-column connection must be capable of sustaining a 0.04 radian interstory drift ratio.**Required flexural strength of connection (kip-ft) = 0.8Mp = 480.00 at story drift angle in (1)**All beam-to-column joints to demonstrate conformance with E3.6b(1) as indicated in E3.6b(2)**See Additional AISC 358 Requirements for Assigned Connection Type.*This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v3B>

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RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Seismic Provisions Member Code Check

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Level 2 Frame No: 2 Member No: 32
 Fy (ksi): 50.00 Size: W18X86
 Frame Type: Special Moment Resisting Frame
 Apply AISC 358 Provisions to Moment Frame

D1.4a Required Strength --- OK

Compression: Max Pu (kip) = 157.22 --- Combination: 1.287 D + 0.500 Lp + 3.000 Dyn5
 Max Pu/φPn = 0.18 OK
Tension: Max Pu (kip) = 4.61 --- Combination: 0.813 D - 3.000 Dyn5
 Max Pu/φPn = 0.00 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

Required tension and compression strength from D1.4a.

Shear in major axis (kip) = 26.92 --- Combination: 1.287 D + 0.500 Lp + 3.000 Dyn5

Shear in minor axis (kip) = 0.13 --- Combination: 1.287 D + 0.500 Lp - 3.000 Dyn8

Moment in major axis (kip-ft) = 363.85 --- Combination: 1.287 D + 0.500 Lp + 3.000 Dyn5

Moment in minor axis (kip-ft) = 1.38 --- Combination: 1.287 D + 0.500 Lp + 3.000 Dyn8

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

See Additional AISC 358 Requirements for Assigned Connection Type.

E3.4c(1) Unbraced Connections --- OK

Joint Above Column is Restrained

Joint Below Column is Restrained

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

For calculation of web limit: Pu (kip) = 157.22 φPy (kip) = 1138.50

Flange b/tf = 7.21 Limit = 7.35 OK

Web h/tw = 33.46 Limit = 51.36 OK

E3.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

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RAM Structural System

Seismic Provisions Joint Code Check

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

02/18/21 21:43:58

Steel Code: AISC341-16 - LRFD

Joint Parameters

Story: Level 2 Frame No: 2 Joint No: 45
 Fy (ksi): 50.00 Column Size: W18X130
 Joint Frame Type: Special Moment Resisting Frame
 Apply AISC 358 Provisions to Moment Frame
 Strain Hardening Factor Cpr = 1.40

E3.4a Moment Ratio --- OK

Col.	Size	M*pc (kip-ft)	Zc (in ³)	Pu (kip)	Combo#
Top	W18X130	1161.65	290.00	73.79	1
Bot	W18X130	1124.45	290.00	132.60	1
Bm.	Size	M*pb (kip-ft)	Mpr (kip-ft)	Muv (kip-ft)	Sh+dc/2 (in)
71	W21X62	1011.64	924.00	87.64	9.65
149	W21X62	1000.30	924.00	76.30	9.65

Sh = Distance face-of-column to location of hinge.

Mpr = *Cpr* x *Ry* x *Fy* x *Zhng*, where *Zhng* = *Z* at hinge.

$\Sigma M^*_{pc} = \Sigma Z_c(F_y - P_u/A_g)$ (kip-ft) = 2286.10 ΣM^*_{pb} (kip-ft) = 2011.94

$\Sigma M^*_{pc} / \Sigma M^*_{pb}$ (Eqn E3-1) = 1.14 > 1.0 OK

E3.4c Stability Bracing at Beam-to-Column Connections --- OK

Joint is restrained in the minor axis at both the top and bottom flange of the moment-frame beams.

Note! RAM Frame assumes that a beam framing into the minor axis of the joint, braces the column at the elevation of both top and bottom flange of the girders connected to the flanges of the column.

E3.6d Connection Required Shear Strength

Bm No.	Col Side	Mpr (kip-ft)	L (ft)	0.8Mp(kip-ft)	V Max (kip)
71	Flange	924.00	20.429	480.00	108.98
149	Flange	924.00	17.429	480.00	117.18

V Max = Max shear from applicable load combinations with *Ecl*

Mpr = *Cpr* x *Ry* x *Fy* x *Zhng*, where *Zhng* = *Z* at hinge.

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- OK With Web Plate / Stiffeners

Bm.	Size	Bf Red. (in)	Z (in ³)	Z hng (in ³)	Mpr (kip-ft)
71	W21X62	0.000	144.00	144.00	924.00
149	W21X62	0.000	144.00	144.00	924.00
Bm.	Mpr (kip-ft)	Sh (in)	V hng (kip)	Mf (kip-ft)	Vpz (kip)
71	924.00	0.00	108.978	924.00	543.93
149	924.00	0.00	117.182	924.00	543.93

Sh = Distance face-of-column to location of hinge.

Vhng = Shear at hinge or col face from applicable load combinations with *Ecl*

Mpr = *Cpr* x *Ry* x *Fy* x *Zhng*, where *Zhng* = *Z* at hinge.

Required Panel Zone Shear Strength (kip) = 953.46

Panel Zone Shear Reduced by *Vc* (kip) = 134.40

Axial Comp. for Panel Zone Calc. (kip) = 132.60 - Combination 1.287 D + 0.500 Lp + 3.000 Dyn5

Panel Zone Strength Without Web Plate (kip) = 457.05 NG

Web Plate Reqd Thickness (in) = 0.857

Web Plate Thickness Used (in) = 0.875

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RAM Structural System

Seismic Provisions Joint Code Check

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Web plate thickness req'd to fillet weld to column flanges (in) = 0.212

Panel Zone Strength With Web Plate (kip) = 963.68 **OK**

E3.6e(2) Panel-Zone of Beam-to-Column Connections: Thickness --- **OK**

No plug welds specified so both column and web plate must meet Equation E3-7

dz (in) = 19.770 wz (in) = 16.900

Column web thickness reqd (in) = 0.407 Thickness provided = 0.670 **OK**Web PL thickness reqd (in) = 0.407 Thickness provided = 0.875 **OK**

358 - 2.4.4 Continuity Plates --- **OK With Web Plate / Stiffeners**

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
71	W21X62	50.00	8.24	0.615	1.37	1.20	Yes
149	W21X62	50.00	8.24	0.615	1.37	1.20	Yes

See Additional AISC 358 Requirements for Assigned Connection Type.

BRACED FRAME DESIGN

NOTE: SEE 2/S7.01 FOR PROTECTED ZONE

Example
Beam Calc

Seismic Provisions Member Code Check Summary

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Criteria

Use frame numbers to designate a single line of bracing

 $C_d/I_e = 4.0$

Apply AISC 358 Provisions to Moment Frame

RBS Strain Hardening Factor $C_{pr} = 1.10$ Other Strain Hardening Factor $C_{pr} = 1.40$

AISC 358 Live Load Factor = 0.50

AISC 358 Snow Load Factor = 0.20

BRBF and SCBF - Required Column Strength based on Larger of Strength and Capacity Limited Seismic Forces.

SAMPLE BRACED FRAME

Frame #1:

Story:Roof Column: 12 Type: Special Concentric Brace Frame -ChevronSize:W14X68 F_y (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story:Roof Column: 13 Type: Special Concentric Brace Frame -ChevronSize:W14X68 F_y (ksi): 50.00

D1.4a Required Strength --- OK

D2.5b Column Splices - Required Strength

F2.6d Column Splices

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

ADDITIONAL CHECK WILL
BE SHOWN IN BRACED
FRAME MEMBER CHECK
SAMPLE CALC.**Story:Roof Beam: 31 Type: Special Concentric Brace Frame -Chevron**Size:W24X131 F_y (ksi): 50.00

F2.4b V- and Inverted V- Braced Frames --- Additional Check Required

F2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Story:Roof Brace: 22 Type: Special Concentric Brace Frame -ChevronSize:HSS3.5X3.5X3/8 F_y (ksi): 50.00

F2.4a Lateral Force Distribution --- OK

F2.5 Bracing Members --- OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

F2.5b(1) Slenderness --- OK

Story:Roof Brace: 23 Type: Special Concentric Brace Frame -ChevronSize:HSS3.5X3.5X3/8 F_y (ksi): 50.00

F2.4a Lateral Force Distribution --- OK

F2.5 Bracing Members --- OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

F2.5b(1) Slenderness --- OK

Story:Level 3 Column: 13 Type: Special Concentric Brace Frame -Chevron

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Size: W14X68 Fy (ksi): 50.00
D1.4a Required Strength --- OK
D2.5b Column Splices - Required Strength
F2.6d Column Splices
F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 3 Column: 14 Type: Special Concentric Brace Frame -Chevron

Size: W14X68 Fy (ksi): 50.00
D1.4a Required Strength --- OK
D2.5b Column Splices - Required Strength
F2.6d Column Splices
F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 3 Beam: 35 Type: Special Concentrically Braced Frame

Size: W18X71 Fy (ksi): 50.00
F2.3 Non-V and Non-Inverted V-Braced Frame - Unbalanced Forces Design Required.
F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 3 Brace: 18 Type: Special Concentric Brace Frame -Chevron

Size: HSS5.5X5.5X3/8 Fy (ksi): 50.00
F2.4a Lateral Force Distribution --- OK
F2.5 Bracing Members --- OK
F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK
F2.5b(1) Slenderness --- OK

Story: Level 3 Brace: 19 Type: Special Concentric Brace Frame -Chevron

Size: HSS5.5X5.5X3/8 Fy (ksi): 50.00
F2.4a Lateral Force Distribution --- OK
F2.5 Bracing Members --- OK
F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK
F2.5b(1) Slenderness --- OK

Story: Level 2 Column: 13 Type: Special Concentric Brace Frame -Chevron

Size: W14X68 Fy (ksi): 50.00
D1.4a Required Strength --- OK
D2.5b Column Splices - Required Strength
F2.6d Column Splices
F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Story: Level 2 Column: 14 Type: Special Concentric Brace Frame -Chevron

Size: W14X68 Fy (ksi): 50.00
D1.4a Required Strength --- OK
D2.5b Column Splices - Required Strength
F2.6d Column Splices
F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

RAM Structural System

Seismic Provisions Member Code Check Summary

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Story:Level 2 Beam: 36 Type: Special Concentric Brace Frame -Chevron

Size:W21X93 Fy (ksi): 50.00

F2.4b V- and Inverted V- Braced Frames --- **Additional Check Required**F2.5a Basic Requirements (D1.1 Moderately Ductile) --- **OK****Story:Level 2 Brace: 10 Type: Special Concentric Brace Frame -Chevron**

Size:HSS5.5X5.5X3/8 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- **OK**F2.5 Bracing Members --- **OK**F2.5a Basic Requirements (D1.1 Highly Ductile) --- **OK**F2.5b(1) Slenderness --- **OK****Story:Level 2 Brace: 11 Type: Special Concentric Brace Frame -Chevron**

Size:HSS5.5X5.5X3/8 Fy (ksi): 50.00

F2.4a Lateral Force Distribution --- **OK**F2.5 Bracing Members --- **OK**F2.5a Basic Requirements (D1.1 Highly Ductile) --- **OK**F2.5b(1) Slenderness --- **OK**

NOTE: SEE 2/S7.01 FOR PROTECTED ZONE

Example
Beam Calc

Seismic Provisions Member Code Check

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

02/18/21 22:20:50

Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Level 2 Frame No: 1 Member No: 36

Fy (ksi): 50.00 Size: W21X93

Frame Type: Special Concentric Brace Frame -Chevron

Criteria

Use frame numbers to designate a single line of bracing

F2.4b V- and Inverted V- Braced Frames --- Additional Check RequiredBeam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

Top Left Brace: Tens (kip) = 404.54

Comp (kip) = 149.61

Post Buck. Comp (kip) = 44.88

Top Right Brace: Tens (kip) = 404.54

Comp (kip) = 149.61

Post Buck. Comp (kip) = 44.88

Bot Left Brace: Tens (kip) = 404.54

Comp (kip) = 149.61

Post Buck. Comp (kip) = 44.88

Bot Right Brace: Tens (kip) = 404.54

Comp (kip) = 149.61

Post Buck. Comp (kip) = 44.88

Unbalanced Beam Forces from V-Braces - F2.3(a) Analysis

Vertical Resultant (kip) = 0.0

Horizontal Resultant (kip) = 0.0

Unbalanced Beam Forces from V-Braces - F2.3(b) Analysis

Vertical Resultant (kip) = 0.0

Horizontal Resultant (kip) = 0.0

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 86.00 Lu Limit = 184.81 **OK**

Required strength of lateral brace along beam (A-6-7) = 0.70 kip

Required stiffness of bracing (A-6-8) = 464.76 kip / Lb

*Cd = 1.0 assumed for eqns (A-6-7/8)**Lb = distance between braces (in)**The brace closest to the point of inflection should be designed for two times these values (Cd=2)***F2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK**Flange b/tf = 4.53 Limit = 9.18 **OK**Web h/tw = 32.31 Limit = 36.05 **OK***Note: Beam axial load from F2.3 Analysis not determined; Web h/tw limit assumed to be $1.57 \sqrt{E/R_y F_y}$.*

SCBF Beam Additional Check

Per AISC Seismic Provision Section F2.3(i) and F2.3(ii)

Roof Beam

Try W24X131	$L_p := 29.33 \text{ ft}$	
$P_n := 1624.91$	Kips	$P_u1 := 3.5$
$M_{nx} := 1387.5$	Kips - ft	$M_u1 := 10.5$
$M_{ny} := 305.6$	Kips - ft	

F2.3(a)

With Unbalanced Load:

Vertical Resultant	Horizontal Resultant
$E_{mv} := 128.3$	$E_{mh} := 163.9$
Kips	Kips

Axial Load

$$P_u := P_{u1} + E_{mh} = 167.4 \text{ Kips}$$

Moment:

$$M_{ux} := M_{u1} + E_{mv} \cdot \frac{L}{4} = 951.26 \text{ Kips - ft}$$

$$M_{uy} := 0 \text{ Kips - ft}$$

Check P_u/P_n

$$\frac{P_u}{P_n} = 0.103$$

$P_r/P_c > 0.2$ Use Equation

$$\frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad H1 - 1a$$

$P_r/P_c < 0.2$ Use Equation

$$\frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad H1 - 1b$$

Interaction Check

$$\text{Int_Check} := \begin{cases} \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} > 0.2 \\ \frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} \leq 0.2 \end{cases}$$

$$\text{Int_Check} = 0.737$$

F2.3(b)

With Unbalanced Load:

Vertical Resultant

$$E_{mv} := 142.1 \text{ Kips}$$

Horizontal Resultant

$$E_{mh} := 150.4 \text{ Kips}$$

Axial Load

$$P_u := P_{u1} + E_{mh} = 153.9 \text{ Kips}$$

Moment:

$$M_{ux} := M_{u1} + E_{mv} \cdot \frac{L}{4} = 1.052 \times 10^3 \text{ Kips} \cdot \text{ft}$$

$$M_{uy} := 0 \text{ Kips} \cdot \text{ft}$$

Check P_u/P_n

$$\frac{P_u}{P_n} = 0.095$$

$P_r/P_c > 0.2$ Use Equation

$$\frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad \text{H1 - 1a}$$

Interaction Check

$$\text{Int_Check} := \begin{cases} \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} > 0.2 \\ \frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{otherwise} \end{cases}$$

$P_r/P_c < 0.2$ Use Equation

$$\frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad \text{H1 - 1b}$$

$$\text{Int_Check} = 0.806$$

W24X131 OK

SCBF Beam Additional Check

Per AISC Seismic Provision Section F2.3(i) and F2.3(ii)

Third Floor Beam

Try W18x71 $L := 29.3$ ft

$P_n := 936$	Kips	$P_{u1} := 12$
$M_{nx} := 547.5$	Kips - ft	$M_{u1} := 376$
$M_{ny} := 92.63$	Kips - ft	

F2.3(b) By inspection, we know this will govern

With Unbalanced Load:

From Member Force in Ram,

$F_{upper} := 206.98$ Kips For Expected Tension of Brace 3.5x3.5x5/16

$F_{lower} := 59.71$ Kips For Expected Post Buckling Comp of Brace 6 x 6 x 3/8

$F_{Diaphragm} := 60$ Kips From Ram Structure, Frame Story Shear

$\theta_1 := 44.705$

$\theta_2 := 43.288$

$$E_{mh} := F_{upper} \cdot \cos(\theta_1 \text{ deg}) - F_{lower} \cdot \cos(\theta_2 \text{ deg}) + F_{Diaphragm} = 163.645 \text{ Kips}$$

Vertical Resultant

$$E_{mv} := 0$$

Kips

Horizontal Resultant

$$E_{mh} = 163.645 \text{ Kips}$$

Axial Load

$$P_u := P_{u1} + E_{mh} = 175.645 \text{ Kips}$$

Moment:

$$M_{ux} := M_{u1} + E_{mv} \cdot \frac{L}{4} = 376 \text{ Kips} - \text{ft}$$

$$M_{uy} := 0 \text{ Kips} - \text{ft}$$

Check P_u/P_n

$$\frac{P_u}{P_n} = 0.188$$

$P_r/P_c > 0.2$ Use Equation

$$\frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad H1 - 1a$$

Interaction Check

$$Int_Check := \begin{cases} \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{if } \frac{P_u}{P_n} > 0.2 \\ \frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) & \text{otherwise} \end{cases}$$

$P_r/P_c < 0.2$ Use Equation

$$\frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) \quad H1 - 1b$$

$$Int_Check = 0.781$$

W18x71 OK

Second Floor Beam

Since the Brace at top and bot are the same(Size and Geometry), Unblanced Load can be ignored, use W18x71

SAMPLE COLUMN CHECK

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RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

Building Code: IBC

Seismic Provisions Member Code Check

02/18/21 22:21:24

Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Level 2 Frame No: 1 Member No: 14
 Fy (ksi): 50.00 Size: W14X68
 Frame Type: Special Concentric Brace Frame -Chevron

Criteria

Use frame numbers to designate a single line of bracing

D1.4a Required Strength --- OK

Max of Adj. Brace Strength and Amp. Seismic Forces with Ω_o Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 513.13 --- Combination: 1.287 D + 0.500 Lp + 1.000 Ecl

Ecl (kips) = 377.03

Ecl: Controlled by F2.3(a) Analysis

Max Pu/ ϕ Pn = 0.79 OK

Tension: Max Pu (kip) = 125.47 --- Combination: 0.813 D + 1.000 Ecl

Ecl (kips) = -200.56

Ecl: Controlled by F2.3(a) Analysis

Max Pu/ ϕ Pn = 0.14 OK

Exception (2) of F2.3 was not checked.

D1.4a(b) – E amplified by Ω_o :

	Load Combination	Pu(kip)	ϕ Pn(kip)	Result
Compression:	1.287 D + 0.500 Lp + 3.000 Dyn7	234.83	647.66	OK
Tension:	0.813 D - 3.000 Dyn7	23.64	900.00	OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.287 D + 0.500 Lp + 1.000 Ecl	513.13
Tension:	0.813 D + 1.000 Ecl	125.47

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

For calculation of web limit: Pu (kip) = 513.13 ϕ Py (kip) = 900.00

Flange b/tf = 6.94 Limit = 7.35 OK

Web h/tw = 27.42 Limit = 42.63 OK

F2.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

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SAMPLE BRACE CHECK

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Seismic Provisions Member Code Check

RAM Structural System

RAM Structural System 17.01.01.05

DataBase: GRCC St

02/18/21 22:21:24

Building Code: IBC

Steel Code: AISC341-16 - LRFD

Brace Parameters

Story: Level 2 Frame No: 1 Member No: 11
 Fy (ksi): 42.00 Size: HSS5.5X5.5X3/8
 Frame Type: Special Concentric Brace Frame -Chevron

Criteria

Use frame numbers to designate a single line of bracing

F2.5 Bracing Members --- OKF2.3 Required Strength

Expected tension strength ($R_y \times F_y \times A_g$) of brace (kip) = 404.54

Expected comp. str. min ($R_y F_y A_g, (1.0/0.877) F_{cre} A_g$) of brace (kip) = 149.64

Expected post-buckling strength of brace (kip) = 44.89

F2.4a Lateral Force Distribution --- OK

The max percentage of lateral load carried by braces for frame # 1 (this story only).

Max Horiz. Brace Force	Load Case	Percent	Result
Compression:	Dyn_ASCE716_SRSS_Y_+E	50.16	<70 OK
Tension:	Dyn_ASCE716_SRSS_Y_+E	50.16	<70 OK

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Flange b/t_f = 12.76 Limit = 14.44 OK

Web h/t_w = 12.76 Limit = 14.44 OK

F2.5b(1) Slenderness --- OK

Max kl/r = 114.71 kl/r limit = 200.00 OK

F2.6 Bracing ConnectionsF2.6c(1) Required Tensile Strength

Strength of the connection shall be the lesser of a or b as follows:

a. Expected ($R_y \times F_y \times A_g$) tensile strength of brace (kip) = 404.54

b. Max force that the brace can transfer to the system - Not Performed

F2.6c(2) Required Compressive Strength

Required compression strength of brace P_c (kip) = 149.64

F2.6c(3a) Rotation Capacity

Brace buckles in direction of member Minor axis, $1.1R_y M_p$ (kip-ft) = 70.61

The design of the gusset plates shall include consideration of buckling.

SAMPLE CONNECTION CHECK

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Bay Size

$L = 14.665$ ft
 $H_{lower} = 13.75$ ft
 $\theta_{lower} = 46.8443614$ deg.
 $H_{upper} = 13.75$ ft
 $\theta_{upper} = 43.1556386$ deg.

Members

Column = W14X68
 BM Stub = W18X71
 Beam = W18X71
 Slab Thk. = 5.5 in

Mat'l:

Gusset = A572-50
 Brace = A500-C SQ & RECT
 Rein. PL = A572-50
 Beam = A992
 Column = A992
 Splice PL = A572-50

Lower Brace

HSS5X5X3/8
Gusset Details:
 $\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 2$ in.
 $t_g = 1$ in.
 $L_w = 18$ in.
 Pull-off Dim = 32.5 in.
 Clip = 0.75 in.
 Over-slot = 2 in.

Gusset Weld Details:

$D_{brace} = 5$
 $D_{column} = 5$
 $D_{beam} = 5$
 $D_{stub-col} = 5$
Reinforcing Details:
 $t_{pl} = 0.75$ in.
 $b = 4$ in.
 $L_w = 10$ in.
 $D_{brace} = 6$

Upper Brace

HSS3-1/2X3-1/2X5/16
Gusset Details:
 $\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 2$ in.
 $t_g = 1$ in.
 $L_w = 18$ in.
 Pull-off Dim = 28.5 in.
 Clip = 0.75 in.
 Over-slot = 2 in.

Gusset Weld Details:

$D_{brace} = 5$
 $D_{column} = 5$
 $D_{beam} = 5$
Reinforcing Details:
 $t_{pl} = 0.75$ in.
 $b = 2.5$ in.
 $L_w = 10$ in.
 $D_{brace} = 6$

Analysis

$P_{uc} = 38$ k
 Drag DF = 1
 $F_{EXX} = 70$ ksi
 Ductility Factor = 1.25
 Gusset K Factor = 0.6
 Distribution Method = UFM

Splice Plate

Axial Load = 276.19 k
 Grav. Shear = 27.5 k
 $t_{pl} = 0.625$ in
 Bolts (DS) = A325-X
 Dia. = 1 in
 Bolt Rows = 4
 Bolt Col. = 3
 Row Spacing = 3 in
 Col. Spacing = 3 in
 le = 2 in
 Gap = 1 in
 Dist = 27.3 in

Connection Transfer Forces

$A_I = 242.1123957$ k
 $N_I = 262.4563474$ k
 $A_{II} = 160.9249126$ k
 $N_{II} = 276.1939357$ k

Upper Gusset

Gusset Edge Forces

Sway Right

$R_y F_y A_g = 228.8$ k
 $H_{uc} = 58.4$ k
 $V_{uc} = 79.3$ k
 $M_{uc} = 89.9$ in-k
 $H_{ub} = 108.5$ k
 $V_{ub} = 77.2$ k
 $M_{ub} = 0.0$ in-k

Gusset Geometry

$L = 29.84$ in
 $LA = 25.23$ in
 $LB = 21.31$ in
 $Lg = 18.31$ in

Sway Left

$1.14 F_{cre} A_g = 41.9$ k
 $H_{uc} = 10.7$ k
 $V_{uc} = 14.5$ k
 $M_{uc} = 16.5$ in-k
 $H_{ub} = 19.9$ k
 $V_{ub} = 14.2$ k
 $M_{ub} = 0.0$ in-k
 $0.3 F_{cre} A_g = 12.6$ k
 $H_{uc} = 3.2$ k
 $V_{uc} = 4.4$ k
 $M_{uc} = 4.9$ in-k
 $H_{ub} = 6.0$ k
 $V_{ub} = 4.2$ k
 $M_{ub} = 0.0$ in-k

Lower Gusset

Gusset Edge Forces

Sway Right

$1.14 F_{cre} A_g = 200.9$ k
 $H_{uc} = 50.0$ k
 $V_{uc} = 71.4$ k
 $M_{uc} = 78.1$ in-k
 $H_{ub} = 96.6$ k
 $V_{ub} = 66.0$ k
 $M_{ub} = 0.0$ in-k
 $0.3 F_{cre} A_g = 60.3$ k
 $H_{uc} = 15.0$ k
 $V_{uc} = 21.4$ k
 $M_{uc} = 23.4$ in-k
 $H_{ub} = 29.0$ k
 $V_{ub} = 19.8$ k
 $M_{ub} = 0.0$ in-k

Sway Left

$R_y F_y A_g = 401.7$ k
 $H_{uc} = 99.9$ k
 $V_{uc} = 142.8$ k
 $M_{uc} = 156.1$ in-k
 $H_{ub} = 193.2$ k
 $V_{ub} = 132.0$ k
 $M_{ub} = 0.0$ in-k

Gusset Geometry

$L = 30.64$ in
 $LA = 26.33$ in
 $LB = 22.39$ in
 $Lg = 19.11$ in

Limit States

<u>Lower Brace</u>	<u>Spec Eqn</u>	<u>R_n</u>	<u>φR_n</u>	<u>R_n/φR_n</u>
Brace Weld	J2-4	401.70	501.16	0.80 <i>OK</i>
Net Sect. Rupt.	J4-2	401.70	551.00	0.73 <i>OK</i>
Shear Yield	J4-3	401.70	979.99	0.41 <i>OK</i>
Shear Rupture	J4-4	401.70	841.29	0.48 <i>OK</i>
Reinf. Weld	J2-4	165.00	167.05	0.99 <i>OK</i>
<u>Lower Gusset</u>				
Tension Yield	J4-1	401.70	1160.31	0.35 <i>OK</i>
Buckling	E3	221.00	1033.83	0.21 <i>OK</i>
Block Shear	J4-3	401.70	1053.75	0.38 <i>OK</i>
<u>Lower Gusset at Col.</u>				
Column Weld	J2-4	240.30	383.79	0.63 <i>OK</i>
Tension Yield	J4-1	128.73	973.80	0.13 <i>OK</i>
Shear Yield	J4-2	142.78	649.20	0.22 <i>OK</i>
<u>Lower Gusset at Beam</u>				
Beam Weld	J2-4	292.43	431.49	0.68 <i>OK</i>
Tension Yield	J4-1	131.98	1150.95	0.11 <i>OK</i>
Shear Yield	J4-2	193.16	767.30	0.25 <i>OK</i>
LWY	J10-2	131.98	707.89	0.19 <i>OK</i>
WC	J10-4	131.98	675.16	0.20 <i>OK</i>
<u>Upper Brace</u>				
Brace Weld	J2-4	228.80	501.16	0.46 <i>OK</i>
Net Sect. Rupt.	J4-2	228.80	335.23	0.68 <i>OK</i>
Shear Yield	J4-3	228.80	817.13	0.28 <i>OK</i>
Shear Rupture	J4-4	228.80	701.47	0.33 <i>OK</i>
Reinf. Weld	J2-4	103.13	167.05	0.62 <i>OK</i>
<u>Upper Gusset</u>				
Tension Yield	J4-1	228.80	1092.81	0.21 <i>OK</i>
Buckling	E3	46.12	982.94	0.05 <i>OK</i>
Block Shear	J4-3	228.80	980.63	0.23 <i>OK</i>
<u>Upper Gusset at Col.</u>				
Column Weld	J2-4	137.21	368.60	0.37 <i>OK</i>
Tension Yield	J4-1	75.94	925.30	0.08 <i>OK</i>
Shear Yield	J4-2	79.26	616.87	0.13 <i>OK</i>
<u>Upper Gusset at Beam</u>				
Beam Weld	J2-4	166.44	416.07	0.40 <i>OK</i>
Tension Yield	J4-1	77.23	1101.61	0.07 <i>OK</i>
Shear Yield	J4-2	108.46	734.40	0.15 <i>OK</i>
LWY	J10-2	77.23	680.75	0.11 <i>OK</i>
WC	J10-4	77.23	655.92	0.12 <i>OK</i>
<u>Beam-Column Connection</u>				
BM-Col Weld	J2-4	142.78	417.63	0.34 <i>OK</i>

Limit States (Cont.)

<u>Beam Splice</u>	<u>Spec Eqn</u>	<u>R_u</u>	<u>ϕR_n</u>	<u>$R_u / \phi R_n$</u>	
Bolt Shear (BM)	J3-6	27.50	635.59	0.04 <i>OK</i>	(Gravity Only)
		277.56	763.85	0.36 <i>OK</i>	(Gravity + Seismic)
Bolt Shear (Stub)	J3-6	27.50	707.60	0.04 <i>OK</i>	(Gravity Only)
		277.56	851.83	0.33 <i>OK</i>	(Gravity + Seismic)
Bolt BRG (BM)	J3-10	27.50	459.72	0.06 <i>OK</i>	(Gravity Only)
		277.56	552.50	0.50 <i>OK</i>	(Gravity + Seismic)
Bolt Tear (BM)	J3-10	27.50	337.61	0.08 <i>OK</i>	(Gravity Only)
		277.56	507.17	0.55 <i>OK</i>	(Gravity + Seismic)
Bolt BRG (Stub)	J3-10	27.50	511.81	0.05 <i>OK</i>	(Gravity Only)
		277.56	616.13	0.45 <i>OK</i>	(Gravity + Seismic)
Bolt Tear (Stub)	J3-10	27.50	375.86	0.07 <i>OK</i>	(Gravity Only)
		277.56	565.59	0.49 <i>OK</i>	(Gravity + Seismic)
PL Buckling	E3	276.19	674.45	0.41 <i>OK</i>	
Shear Yield	J4-3	27.50	487.50	0.06 <i>OK</i>	
Flexural Yield	F11-1	820.23	2376.56	0.35 <i>OK</i>	
Shear Rupture	J4-4	27.50	310.78	0.09 <i>OK</i>	
Flexural Rupture	F9-4	820.23	1751.95	0.47 <i>OK</i>	
Ductility	10-3	1.25	1.88	0.66 <i>OK</i>	
Rotation	F2.6b	0.23	1.00	0.23 <i>OK</i>	

Bay Size

$L = 14.67$ ft
 $H_{lower} = 13.75$ ft
 $\theta_{lower} = 46.84$ deg.
 $H_{upper,R} = 13.75$ ft
 $\theta_{upper,R} = 43.16$ deg.
 $H_{upper,L} = 14.58$ ft
 $\theta_{upper,L} = 44.84$ deg.

Members

Beam = W21X93
 Slab Thk. = 5.5 in

Mat'l:

Gusset PL = A36
 Brace = A500-B SQ & RECT
 Reinfr. PL = A572-50
 Beam = A992
 Column = A500-B SQ & RECT

Doubler PL Details:

$t_{PL} = 0.3125$ in
 $h = 8$ in

Upper Brace

HSS5-1/2X5-1/2X3/8

Gusset Details:

$\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 1$ in.
 $t_g = 0.75$ in.
 $L_w = 20$ in.
 Pull-off (R) = 32 in.
 Pull-off (L) = 32 in.
 Clip = 1 in.
 Over Slot = 2 in.
 $D_{brace} = 5$
 $D_{Beam} = 5$

Brace Reinforcing Details:

$t_{PL} = 0.5$ in.
 $b = 5$ in.
 $L_w = 9$ in.
 $D_{brace} = 6$

$L = 0$ in
 $D_{brace} = 4$

Lower Brace

HSS5-1/2X5-1/2X3/8

Gusset Details:

$\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 1$ in.
 $t_g = 0.75$ in.
 $L_w = 20$ in.
 Pull-off = 32 in.
 Clip = 1 in.
 Over Slot = 2 in.
 $D_{brace} = 5$
 $D_{Beam} = 5$

Brace Reinforcing Details:

$t_{PL} = 0.5$ in.
 $b = 5$ in.
 $L_w = 9$ in.
 $D_{brace} = 6$

$L_{min} = -20.459$ in

Analysis

Welds $F_{EXX} = 70$ ksi
 Weld Ductility Factor = 1.25
 Gusset K Factor = 0.65
 Drag DF = 0.5
 Doubler PL OK? Yes

Status

DESIGN OK!

Upper Gusset

Gusset Edge Forces

Left Brace:

$R_v F_v A_g = 443.072$ k
 $1.14 F_{cre} A_g = 248.1949$ k
 $0.3 F_{cre} A_g = 74.45848$ k
 Right Brace:
 $R_v F_v A_g = 443.072$ k
 $1.14 F_{cre} A_g = 262.8884$ k
 $0.3 F_{cre} A_g = 78.86651$ k
 $N_u = 128.0459$ k
 $V_u = 499.212$ k
 $M_u = 5391.49$ in-k
 $N_{u,eq} = 437.9191$ k
 $N_u = 250.5512$ k
 $V_u = 376.0178$ k
 $M_u = 4060.993$ in-k
 $N_{u,eq} = 483.9548$ k

Gusset Geometry

$L_{Left} = 32.51$ in
 $L_{Right} = 33.03$ in
 $LA = 69.60$ in
 $LB = 20.67$ in
 $Lg, max = 18.74$ in

Lower Gusset

Gusset Edge Forces

$R_v F_v A_g = 443.072$ k
 $1.14 F_{cre} A_g = 262.8884$ k
 $0.3 F_{cre} A_g = 78.86651$ k
 $N_u = 123.2424$ k
 $V_u = 514.997$ k
 $M_u = 5561.967$ in-k
 $N_{u,eq} = 439.7177$ k
 $N_u = 249.1102$ k
 $V_u = 380.7533$ k
 $M_u = 4112.136$ in-k
 $N_{u,eq} = 483.0902$ k

Gusset Geometry

$L = 33.03$ in
 $LA = 70.30$ in
 $LB = 20.53$ in
 $Lg = 18.74$ in

Limit States

	<u>Spec</u>				
<u>Upper Brace</u>	<u>Eqn</u>	<u>R_n</u>	<u>φR_n</u>	<u>R_n/φR_n</u>	
Brace Weld	J2-4	443.1	556.8	0.80	OK
Net Sect. Rupt.	J4-2	443.1	515.4	0.86	OK
Shear Yield	J4-3	443.1	1078.8	0.41	OK
Shear Rupture	J4-4	443.1	874.5	0.51	OK
Reinf. Weld	J2-4	137.5	150.3	0.91	OK
<u>Upper Brace-Gusset</u>					
Tension Yield	J4-1	443.1	694.8	0.64	OK
Buckling	E3	289.2	588.2	0.49	OK
Block Shear	J4-5	443.1	665.4	0.67	OK
<u>Upper Gusset-Beam</u>					
Shear Yield	J4-3	499.2	1127.5	0.44	OK
Tension Yield	J4-1	437.9	1691.2	0.26	OK
		484.0	1691.2	0.29	OK
Flexural Yield	F11-1	5391.5	29424.9	0.18	OK
Interaction		0.23	1.0	0.23	OK
		0.17	1.0	0.17	OK
Beam Weld	J2-4	830.1	1228.3	0.68	OK
		766.1	1308.8	0.59	OK
WLY	J10-2	437.9	2225.6	0.20	OK
		484.0	2225.6	0.22	OK
WC	J10-4	437.9	1772.9	0.25	OK
		484.0	1772.9	0.27	OK
<u>Lower Brace</u>					
Brace Weld	J2-4	443.1	556.8	0.80	OK
Net Sect. Rupt.	J4-2	443.1	515.4	0.86	OK
Shear Yield	J4-3	443.1	1078.8	0.41	OK
Shear Rupture	J4-4	443.1	874.5	0.51	OK
Reinf. Weld	J2-4	137.5	150.3	0.91	OK
<u>Lower Brace-Gusset</u>					
Tension Yield	J4-1	443.1	694.8	0.64	OK
Buckling	E3	289.2	588.2	0.49	OK
Block Shear	J4-5	443.1	665.4	0.67	OK
<u>Lower Gusset-Beam</u>					
Shear Yield	J4-3	515.0	1138.8	0.45	OK
Tension Yield	J4-1	439.7	1708.3	0.26	OK
		483.1	1708.3	0.28	OK
Flexural Yield	F11-1	5562.0	30022.3	0.19	OK
Interaction		0.23	1.0	0.23	OK
		0.17	1.0	0.17	OK
Beam Weld	J2-4	846.5	1234.7	0.69	OK
		768.9	1319.2	0.58	OK
WLY	J10-2	439.7	2246.0	0.20	OK
		483.1	2246.0	0.22	OK
WC	J10-4	439.7	1787.7	0.25	OK
		483.1	1787.7	0.27	OK
<u>Beam</u>					
Shear Yield	G2-1	315.6	450.8	0.70	OK
		234.4	450.8	0.52	OK
Doubler Weld	J2-4	7.52	8.35	0.90	OK

END